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1 Introduction

1.1 History

The proposed Reno Railroad Corridor is the culmination of many years of extensive analysis. For over 60 years, the City of Reno, in cooperation with various state and federal agencies, has investigated alternatives to reduce the adverse effects of railroad traffic in the downtown City of Reno area. This process included a proposal in 1936 by the United States Bureau of Public Roads to elevate the railroad. In response, the City of Reno City Engineer suggested instead that the railroad remain in its current location and that it be lowered below street level. This depressed railroad concept was intended to be less disruptive to the character of the downtown area than an elevated structure, which would create a barrier through the City of Reno. In a 1942 report, The City of Reno Chamber of Commerce subsequently endorsed the depressed trainway project, at a cost estimate of $1.4 million. Updated reports endorsing a depressed railroad corridor were prepared in 1944, 1968, 1972, 1976, and 1980. Notwithstanding these endorsements, a combination of engineering infeasibility, prohibitive costs and a negative political climate continued to preclude construction of a depressed trainway through the City of Reno.

More recently, in 1996, approval of the Union Pacific/Southern Pacific Railroad merger precipitated renewed discussion of the railroad corridor through the central portion of the City of Reno. The Final Mitigation Plan (U.S Surface Transportation Board 1998) for the merger estimated that the railroad traffic through the corridor would grow substantially over current levels. A depressed trainway was then proposed by the City of Reno as a means to address the adverse effects of existing and anticipated railroad traffic.

A Memorandum of Understanding (MOU) between the Union Pacific Railroad and the City of Reno was executed in December 1998. The MOU specified UPRR’s funding contribution and involvement in the project. In addition, the MOU stipulated the planning, construction, and operating relationship between the City of Reno and the UPRR. Furthermore, the parties agreed that the City of Reno would withdraw its appeal pending before the United States D.C. Circuit Court of Appeals in the City of Reno’s litigation against the Surface Transportation Board and the Union Pacific Railroad. The outcome of these negotiations defined a City of Reno project that became known as the Reno Transportation Rail Access Corridor (ReTRAC) Project.

In June 1999, a federally sponsored process (called the Reno Railroad Corridor to distinguish it from the City of Reno’s ReTRAC project) to develop preliminary engineering and environmental documentation for the project was initiated. The Federal Highway Administration (FHWA) is the federal lead agency for purposes of the National Environmental Policy Act (NEPA). The Nevada Department of Transportation (NDOT) is the contracting authority for the process. Cooperating parties include the City of Reno, Washoe County, the Federal Railroad Administration (FRA), and Union Pacific Railroad (UPRR). It is the intention of all parties concerned that modern, cost-effective engineering techniques may finally enable the depressed trainway project to be built.

The Reno Railroad Corridor would be partially funded through the Federal Highway Administration (FHWA) and would be subject to other regulatory requirements.

1.2 Overall Site Conditions

The Union Pacific Railroad tracks extend through the heart of downtown Reno in an east-west direction, in close proximity to local businesses, which include light industrial warehousing, densely populated apartments, hotels, and casinos. Construction of a depressed railroad corridor presents engineering and project coordination challenges in the areas of utility relocation, business coordination, property acquisition, and construction of a watertight trench section.

Figure 1-1 Downtown Reno (Looking East)

Coordination is complicated by the presence of 10 different utility types, 5 large Casinos, several smaller retail businesses and warehouses, and numerous individual property owners. These special interests, combined with anticipated challenging construction, require selection of an alternative that minimizes impacts to existing facilities. The selected alternative will limit shoofly operation, noise, and traffic impacts; can be constructed quickly; and reduce the number of underground impacts. Analysis of these criteria, combined with project costs, resulted in final trench construction recommendations.

The Reno Railroad Corridor involves complex geological, environmental, structural, and commercial issues. This report analyzes how effective engineering practice can be used to resolve these issues.

1.2.1 Project Zones

The project starts at West Second Street, passes through downtown to just east of Sutro Street. It is subdivided into two main design conditions for the trench section. These sections, one above and one below the groundwater table, are known as Zone 1 and 2, respectfully.

Zone 1 is located from the western terminus of the depressed trainway to just west of Vine Street and from just east of Wells Avenue at the Rusty Spike Substation to the eastern terminus, for a total length of 4,420 feet (0.8 miles). The trench section starts at grade and continues subterranean to a maximum depth of 29 feet, measured from the top of the trench to the top of the proposed rail. This section is completely above the design groundwater table. Zone 1 straddles Zone 2, with the eastern end starting just east of the Rusty Spike Substation and continuing under Wells Avenue to Sutro Street. The adjacent facilities include manufacturing, distribution, light commercial, and parking lots.

Zone 2 is positioned between the eastern and western Zone 1 sections. The limits of this zone start just west of Keystone Avenue and extends under Vine Street, Washington Street, Ralston Street, Arlington Avenue, West Street, North Sierra Street, Virginia Street, Center Street, Lake Street, and Evans Avenue to end near the Rusty Spike Substation, for a total length of 6,680 feet (1.3 miles). Hotel, casino, and parking facilities are arranged adjacent to the proposed trench section. In Zone 2, the maximum depth from the top of the proposed trench to top of the proposed rail is 29 feet, which is 6 feet below the design groundwater elevation. The Fitzgerald's parking garage, Fitzgerald's Rainbow Bridge and the historic buildings between North Center Street and Lake Street require underpinning during construction, necessitating special consideration of additional lateral loads imposed on the trench walls or varying construction details.

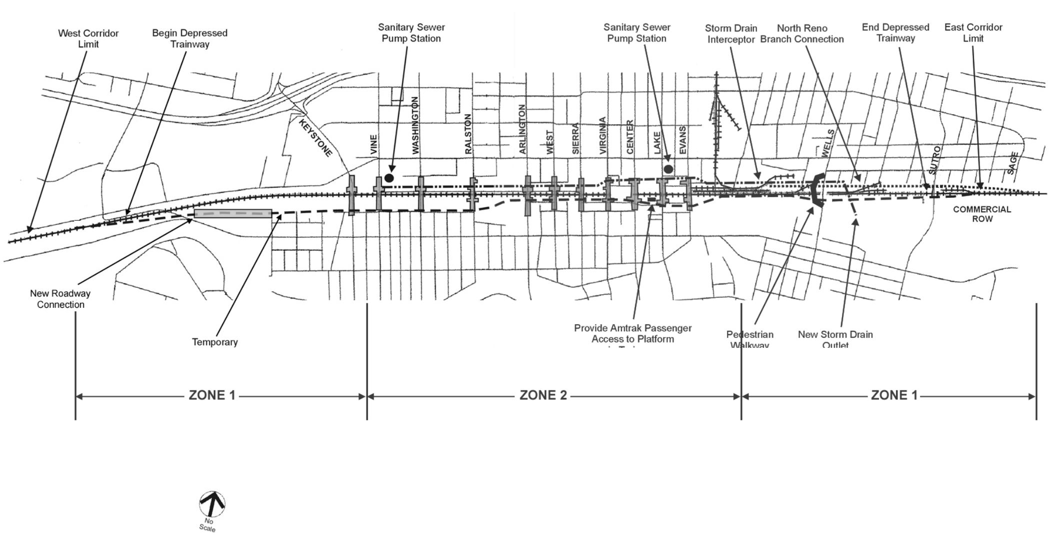


Figure 1-2 Plan View of Project Limits

1.2.2 Shoofly Alignment

Allowing construction while operating the UPRR is accomplished by utilizing a shoofly (a temporary alignment used for rail traffic). For Stage 2 of the construction, the shoofly would be constructed on the south side of the alignment at West Second Street and continue parallel to the existing alignment to rejoin the main track at North Arlington. For Stage 3 of the construction (Central Reno), the shoofly would be located along Commercial Row between North Arlington and North Center Street. Then the shoofly would separate from the main alignment again at North Center Street and run on the south side of the main tracks to the existing Rusty Spike Substation, where it would pass within 16 feet of the proposed trench walls. Past the substation, the alignment would veer south and parallel the trench, joining the existing UPRR tracks 900 feet (0.2 miles) past Sutro Street. The total length of shoofly is approximately 11,500 feet (2.2 miles) [see Appendix C].

1.2.3 Proposed Trench Section

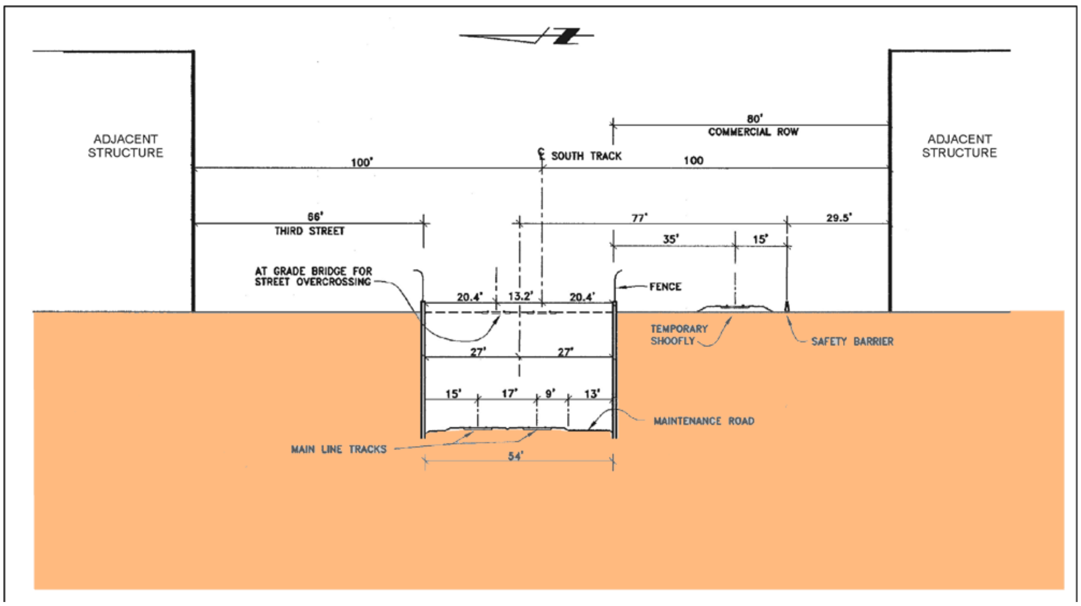


Figure 1-3 Trench Section

As seen in the Figure 1-3, the proposed trench section contains a double-track and maintenance road within its clear width of 54 feet. During the Stage 3 construction along Commercial Row, the shoofly tracks would be within 35 feet of the adjacent trench wall, and the trench section would be within 56 feet of nearby structures on the north and 80 feet of nearby structures on the south. In the vicinity of the Fitzgeralds's parking garage, the underpinned parking structure would be supported by the trench walls.

1.2.4 Soil

The soils along the alignment were explored by a series of borings as described in the *Kleinfelder Geotechnical Report*, dated May 9, 2000. Soils consist predominantly of coarse granular outwash deposits ranging from sand to boulder size particles. These deposits are characterized by a silty to clayey matrix, although zones of deposits with fewer fines are also abundant. Discontinuous layers or lenses of finer grained silty and clayey soils are interbedded in the coarse outwash materials.

The soils are generally dense and uncemented. The amount of cobbles and boulders (defined as particles larger than 3 inches in diameter) varies throughout the alignment and is estimated to be about 30 to 40 % based on large diameter (39 inch) borings drilled in the central portion of the alignment. The presence of cobbles and boulders resulted in a very slow rate of drilling for the large diameter borings (drilled with a Soil Mec R722HD rig). Typically, it took 2 to 3 days to complete a boring 55 feet deep. However, the SONIC drilling for smaller diameter sampling (6- to 8-inch diameter) accomplished 135 feet in depth per day.

The presence of boulders has the greatest impact on alternative types of wall construction. Boulders up to 6 feet in diameter have been reported in excavations in the downtown area and one boulder that size was encountered in the recent borings. Six of the small diameter borings encountered 3- to 5-foot boulders and another six borings encountered 2-to 3-foot boulders. It should be noted that approximately one-third of the small diameter borings did not encounter any boulders.

There does not appear to be any clear geologic pattern of higher and lower concentrations of boulders. Several of the small diameter borings in the central third of the alignment did not encounter boulders. However, the large diameter boring in the same area encountered boulders up to 2 to 3 feet throughout the entire depth of the boring and was the most difficult of the three large diameter borings to drill. The deeper borings do indicate fewer boulders at depth, however there is no apparent pattern within the planned depth of the excavation.

1.2.5 Groundwater

Groundwater was encountered in wells at the project site at depths ranging from 19- to 35-feet below the ground surface. As indicated in alignment Alternative 2 (described in the *Draft Reno Railroad Corridor Environmental Impact Statement*) and the design groundwater profile, the trench would extend below the groundwater table starting at just east of Washington Street to the Rusty Spike Substation. The largest hydrostatic head encountered between these two points is between Virginia Street and Center Street, with a head (due to the design groundwater elevation[1]) measured over the top of proposed rail profile of approximately 6 feet. Variations in this head are caused by seasonal and long-term variations. Typical seasonal variations in the groundwater table of 2- to 3-feet should be expected. Long-term climatic variations (associated with periods of drought or heavy runoff) are estimated to cause changes of up to 6 feet in groundwater levels.

A pump test and 10 slug tests measured the hydraulic conductivity (permeability) of the soils in the zone of the alignment. The results vary from a low of

[1] Third Party Review of Reno Rail Corridor Groundwater Investigation. October, 17, 2000. URS.

$9x10^{-4}$ cm/sec to a high of $1.4x10^{-1}$ cm/sec. The pump test indicates a value of about $1x10^{-1}$ cm/sec with a transmisivity ranging from 5,100 ft^2/day to 21,000 ft^2/day. Analysis of the data from the pumping test also suggests that some dewatering of the upper aquifer occurred. This is based on the decreased flow rate to the pumped well over time given a steady-state head. However, the amount of dewatering realized is insignificant compared to the site and proposed work. With traditional bottom-up construction, these large values of hydraulic conductivity and minor dewatering require excessive expenditures and expose the project to environmental risks. Therefore, top-down construction techniques are recommended to reduce costs and environmental risks to acceptable levels in the region of the alignment that is below the groundwater table (Zone 2). Analysis of the data from the pumping test also suggests that some dewatering of the upper aquifer occurred. This is based on the decreased flow rate to the pumped well over time given a steady-state head.

1.3 Downtown Atmosphere

The downtown hotel/casino area (Figure 1-4) is a major source of revenue for the City of Reno. Noise, traffic disruption, business access disruption, and duration of construction would reduce the attractiveness of the area to customers and may have a major fiscal impact on businesses in the downtown area. In this regard, it is critical to minimize the duration of the Commercial Street shoofly. Construction techniques that may be relatively costly but minimize adverse effects have clear advantages over other less costly techniques that have larger construction zone impacts.

Likewise, techniques reducing the single-track railroad operations of the central shoofly have obvious advantages regarding operations of the UPRR.

Figure 1-4 Downtown Atmosphere

2 General Concepts

Project success is defined by stabilizing the wall excavation at a vertical slope and minimizing groundwater infiltration through the walls and invert of the finished trench. Several methods and techniques are available to address these requirements. Suitability of a given technique depends upon several factors. Each of these factors and the comparison approach used are discussed in detail in the following sections. To determine the applicability of each technique for the Reno Railroad Corridor, construction constraints must be identified for each method.

The exorbitant costs associated with pumping and treating large volumes of water, combined with the adverse environmental impacts associated with mitigation of water infiltration, favor a top down method of construction. A top down method allows the contractor to build the structural elements with equipment at a working surface above the groundwater table. In this scenario, the walls can be constructed, the center of the channel can be excavated to just above the groundwater table and then the invert can be constructed. The method described requires only limited site dewatering, thereby avoiding excessive costs and negative environmental effects.

The presence of boulders in the soil also presents a difficult, but surmountable, engineering challenge to construct the trench with common techniques while controlling final costs.

2.1 Analysis Subjects

The Analysis of Wall and Invert Systems Report examines multiple construction techniques to overcome site-specific challenges. Each technique was identified based on its successful application at sites with similar conditions. However, due to their dramatically different characteristics, each method only presents a possible solution to a particular portion of the project. The purpose of the document is to identify each of the methods considered and present its particular applicability, as well as any shortcomings. To accomplish this task, criteria to assess the limitations of each technique were identified, as follows:

- Applicability to Soil Conditions
- Stability of Wall Construction
- Groundwater Control
- Abutment Related Issues
- Duration of Construction
- Traffic and Noise Impact
- Right-of-Way Impact
- Aesthetics
- Cost
- History of Successful Application
- Application

Each of the aforementioned criteria will be assessed for both wall and invert construction methods, where applicable.

Wall Construction Methods

Wall construction methods have been divided into two distinct categories: 1) walls constructible in all areas within the alignment [Zone 1 or 2] and 2) walls constructible only in regions above groundwater [Zone 1]. Other construction methods that were examined and later dismissed from further consideration (ground freezing, sheetpiling, and soil mixing) were not discussed in detail; however, brief explanations of why these systems were deemed impractical are provided in Section 7. The following is a list, in order of appearance in the report, of all methods that are discussed in detail:

Methods Applicable to All Areas (Zone 1 or 2):

- Slurry-Diaphragm walls
- Jet Grouting
- Permeation Grouting
- Secant/Tangent Piles

Methods Applicable to Dry Regions Only (Zone 1):

- Cantilever Walls
- Mechanically Stabilized Earth
- Micropiling
- Soil Nailing
- Soldier Piles and Lagging
- Stress Walls

Wall Secondary Lateral Support Techniques

To further optimize effectiveness of the primary wall construction techniques, two types of secondary excavation support techniques are often used, struts and grouted ground anchors (tiebacks).

The selection of secondary techniques depends upon several factors, including: 1) available rights-of-way, 2) aesthetics, 3) cost, and 4) soil conditions. Secondary techniques, which can be utilized to improve primary techniques, are typically more cost effective than modifying the structural section of the primary system.

Invert Construction Methods

Since invert construction is only required in regions below the groundwater, all of the following techniques were tested against Zone 2 criteria. The following is a list, in order of appearance in the report, of the methods that are discussed in detail:

- Jet Grouting
- Permeation Grouting
- Cast-in-Place Slab

3 Presentation of Analysis Criteria

The following describes each of the analysis subjects. Outlined in each of the following sections is a description of the process and parameters used in the detailed examination of each wall and invert system. Each construction method is analyzed against the criteria defined in the following subsections. These subsections appear in the detail of Sections 2 through 5 in the same order as presented below.

3.1 Applicability to Soil Conditions

Wall and invert systems are available for every soil type. However, certain systems are not compatible with, or pose particular difficulties in, certain geological conditions. Because of the presence of boulders in the soil, the Reno Railroad Corridor project poses a challenge for traditional wall and invert construction methodology. Factors that must be taken into consideration to determine the suitability of specific wall and invert types for the existing stratum include: 1) presence of large obstructions, 2) cohesionless constituents, 3) difficulty of excavation, and 4) handling of contaminated materials. In the following report, each system is evaluated for specific soil conditions to determine its compatibility with the site.

3.2 Stability of Wall Construction

Special geotechnical consideration is required for two of the proposed wall construction techniques, namely slurry-diaphragm and secant-tangent pile walls. Excavations required for slurry-diaphragm and secant-tangent pile walls are typically held open using bentonite slurry (a mixture of native soil, bentonite, and water) until they can be backfilled with structural concrete. The outward pressure provided by the slurry, on the sides of the excavation, resists the lateral forces that cause caving. However, a major concern of applying this technique to the excavation in the Reno Railroad Corridor is the stability of this system when employed adjacent to live heavy rail.

During the construction of this proposed trench system, Union Pacific will be operating live rail. This operation may be as near as 16 feet from the slurry filled excavation. Examination of this operation was required to determine the susceptibility of the trench to caving and the magnitude of vertical displacements under the UPRR track. Appendix D presents the results from a finite-difference analysis of static and dynamic loading of the trench system from the adjacent train ("Vibration Impacts on Planned Trench Construction", Kleinfelder, January 2001). These results indicate that the trench system is stable and the vertical deflections under the tracks are within acceptable limits to the Union Pacific Railroad.

3.3 Groundwater Control

Controlling groundwater during construction and after completion of the project is essential due to costs associated with dewatering during construction and pumping/treating groundwater that could infiltrate the final trench. Any impact on groundwater in the downtown City of Reno area has potential to affect the quality of the adjoining Truckee River. Therefore, in the region that the final trench is under the groundwater table, the investigation of permeability and proper application of trench structural systems is paramount.

All wall and invert systems to be used in Zone 2 are evaluated based on their ability to create an effective barrier for groundwater infiltration. Furthermore, dewatering during construction must be minimal. Therefore, each system is segregated by its ability to be constructed in the region of possible groundwater infiltration.

Groundwater infiltration may be calculated based on soil permeability. The coefficient of permeability, also termed hydraulic conductivity of soil (k), given in terms of length per unit of time. This coefficient is used to calculate seepage of water through a material mass. Calculations for each of the proposed systems have been completed and included in Appendix E of this report. Those wall systems that provide a positive barrier to groundwater are favored in this analysis.

3.4 Abutment Related Issues (Wall Systems Only)

The proposed Reno Railroad Corridor contains eleven overhead local street crossings for pedestrian and highway vehicle traffic. Vehicles and pedestrians apply added loads, including lateral loads due to seismic response, to the crossing structures. All of these additional loads must be supported on substructure members. Typical bridge construction uses abutments to support these loads and provide a path to transfer these superstructure forces to the soil. Since the abutments may be either pile supported, rest on spread footing, or employ a load path through the trench wall system, the abutments are assumed to be located either adjacent to or directly on the earth retaining wall system.

Each wall system has been examined to determine its capability to resist both vertical and horizontal loads. The analysis of the systems includes a detailed evaluation of the implications of use at bridge crossing locations, possible use of secondary lateral support systems, associated costs, and construction schedule impacts.

The invert system is not influenced by structural abutment issues, therefore this criterion will not be used in the analysis of the proposed systems.

3.5 Duration of Construction

It is a goal of the Reno Railroad Corridor project to complete construction of the Central Reno portion (bounded by Arlington and Evans Avenues) of the trench in no more than a 24-month timeframe. The length of time required for each construction procedure was examined to determine its viability.

To provide a complete analysis, the construction period is separated into four distinct stages: 1) utility relocation, 2) construction of the east and west trench ends, 3) construction of the downtown corridor, and 4) completion of the project. For the purposes of this report, the phases before and after construction of the trench facilities were the same for all options, and were not discussed.

The construction of the trench facilities depends on the selected structural system, installation techniques, and equipment and material availability. Through collaboration with industry experts, the analysis of each system provides an estimate of construction time per square foot face of wall.

3.6 Traffic and Noise Impact

The Reno Railroad Corridor is located in the central portion of the City of Reno adjacent multi-story casinos, hotels, shops, and other ‘round-the-clock’ business enterprises. Noise or traffic during shoofly operation, during construction of the trench, and after completion of the project may impact City of Reno residents, business owners, and visitors.

Construction of the trench, in Central Reno, is estimated to require approximately 24 months. During this time, traffic from construction vehicles and noise from associated operations would have an effect on the neighboring community. Noise effects are dependent on duration of activity and required equipment. Traffic would be affected by the use of construction staging areas, installation methods, and general activities required for construction. These factors are discussed in detail in each of the structural system examinations, since particular installations would be able to substantially reduce construction noise and traffic impacts on the neighboring community.

Also, the existing roads are designed for a finite serviceable life based on traffic volumes and associated vehicle weight. Construction activities on these local roads will affect their serviceable life. Quantifying these impacts at this preliminary stage of investigation is difficult; therefore, they are not discussed in this report. However, it would be appropriate to include these impacts in the final design discussions.

3.7 Right-Of-Way Impact (Wall Systems Only)

The final construction would consist of walls and a trench supporting soil, thereby allowing Union Pacific's railroad tracks to be depressed below the existing street grade for a distance of approximately 2.1 miles to a maximum depth of approximately 29 feet (measured from original ground to the rail profile). These walls may include additional lateral supporting systems consisting of either ground anchors or struts. The installation methods for and final configurations of the proposed wall systems may violate either temporary construction easements or permanent property boundaries delineated as rights-of-way. The need for secondary systems for each wall type is examined in the body of the report. Ground anchor systems, described later in this report, have the largest right-of-way impacts. Therefore, to completely analyze all options, right-of-way or easement impacts must be evaluated; including those imposed by secondary lateral support systems.

Temporary construction easements are boundaries created by local authorities to restrict construction impacts to well defined areas. In the case of the Reno Railroad Corridor, the City of Reno, Union Pacific Railroad, or local utility companies have established these easements.

Right-of-way boundaries are determined in legal documentation held by landowners. These boundaries indicate ownership or right of entry. Property owner rights may be violated if permanent fixtures are installed across right-of-way boundaries. Installation of ground anchors into any wall system to resist higher than normal lateral forces may violate these agreements. In some cases, temporary anchors could be constructed and later de-tensioned and removed, or the tendons may be left in the soil. In the case of temporary installation, the only permanent impact is the requirement to cut through the remaining tendon, if left in the soil, for subsurface utility installation or adjacent building construction. If no ground anchors were required, there would be no additional right-of-way impact.

During development of the *Draft Reno Railroad Corridor Environmental Impact Statement*, an examination of the required underground easements for grouted ground anchors was conducted. At that time, desired easements were established throughout the rail alignment. Starting at West Second Street and extending 1,950 feet to the east (Station 2559+00), the proposed underground easement reaches 15 feet beyond the inside face of the trench on either side. From Station 2559+00 to 2568+00, just West of Keystone Avenue, the proposed underground easement is 30 feet beyond the trench wall face. Continuing along the alignment to 2632+00, 375 feet west of Wells Avenue (through Central Reno), the proposed underground easement is 50 feet on either side of the trench. The proposed underground easement is 34 feet on both sides of the trench from Station 2632+00 to Station 2641+00 (a distance of 900 feet). Finally, from Station 2641+00 to Station 2651+20, the eastern terminus, the proposed underground easement is 15 feet from the trench wall face.

Since the invert system will be contained within the plan limits of the wall system, invert construction feasibility is not influenced by right-of-way. Therefore, right-of-way constraints are not used to evaluate invert systems.

In addition to underground easements, it is anticipated that the at-grade construction easement will allow for a width of 45 feet to the north or south of the inside face of the proposed trench wall.

3.8 Aesthetics (Wall Systems Only)

The extensive wall systems used in the Reno Railroad Corridor would be seen by hundreds of thousands of people per year, leaving a lasting aesthetic impression. Since the large faces of any wall are susceptible to graffiti, soot, dirt, and other forms of degradation, providing a neat, visually appealing wall surface is often a challenging task. Vegetation, artwork, facades, and other measures are available to detract from the harsh appearance of such retaining structures.

To ensure selection of the best wall system for the Reno Railroad Corridor, this report examines the adaptability of each wall concept to aesthetic treatments, describing in detail the possible decorative elements, installation procedures, and scheduling impacts, and costs.

The invert system will not be visible through the track work of the UPRR. Therefore, aesthetics for the invert system were ignored.

3.9 Conceptual Calculations

Construction of the wall and invert systems for the Reno Railroad Corridor would present geotechnical, environmental tolerance, and structural acceptability challenges. To address the structural concerns within the narrow specifications, conceptual calculations were made for each system to evaluate structural stability and constructibility.

The conceptual calculations for the wall systems were performed for two distinctly different scenarios. The wall systems that have been found, through application of various criteria, to be viable for Zone 1 were analyzed based on the following soil and materials design parameters (*Geotechnical Engineering Report, Proposed Reno Railroad Corridor EIS, Reno, NV*, Prepared by Kleinfelder, May 19, 2000):

<u>Soil:</u>

Parameter	Value	Description
γ_{soil}	115 pcf	Unit weight of dry soil
μ	0.45	Static coefficient of friction
Pa	35 psf/ft	Equivalent active fluid pressure above the water table
P_{eq}	450 psf/ft	Equivalent passive fluid pressure above the water table

<u>Materials:</u>

Parameter	Value	Description
γ_{conc}	155 pcf	Unit weight of reinforced concrete
f_y	60 ksi	Specified yield strength of steel

In addition, the wall and invert systems in Zone 2 were scrutinized by the following criteria:

Soil:

Parameter	Value	Description
ϕ	34°	Internal angle of friction
γ_{soil}	115 pcf	Unit weight of soil
γ'_{soil}	130 pcf	Unit weight of saturated soil
Pa	80 psf/ft	Equivalent active fluid pressure below the groundwater table
P_p	250 psf/ft	Equivalent passive fluid pressure below the groundwater table

Materials:

Parameter	Value	Description
γ_{conc}	155 pcf	Unit weight of reinforced concrete
f_y	60 ksi	Specified yield strength of steel

Loading criteria used in the analysis of the proposed systems includes allowances for surcharges due to rail traffic, buildings, and parking facilities. These loads are particularly important at locations where the construction shoofly or a parking lot is within a lateral distance equal to or less than the adjacent trench wall height. Key locations for these loads exist at the Rusty Spike Substation (shoofly surcharge) and between Ralston and North Arlington (parking facilities).

3.10 Cost

Providing accurate cost data during preliminary engineering is difficult. However, the information contained in this report, combined with communication with competent contractors and geotechnical and structural experts, has permitted the specification of approximate costs per square foot of each system. Specifically, the information presented in this report on each viable invert and wall type includes an approximate construction cost for distinct sections of the trench, namely Zones 1 and 2 (where applicable). These values are used to rank the different wall and invert systems for final recommendations.

3.11 History of Successful Application

To establish the acceptability of particular structural systems for use in the Reno Railroad Corridor, successful projects with characteristics similar to the Reno

Railroad Corridor are described in the body of this report. The systems used for these similar projects are discussed and project references are provided.

3.12 Application

The Reno Railroad Corridor consists of depressing the Union Pacific Railroad tracks beneath the existing ground surface for a maximum depth of approximately 29 feet measured from the top of the trench to the proposed rail profile. Such a depression would require a sophisticated invert system and retaining structures, each with its own challenges, advantages, and applicability to the project. To determine the applicability of each system to this project, selection criteria were examined, including: 1) regions of applicability, 2) the installation procedures, 3) functional performance, and 4) materials and equipment availability.

The regions of applicability for each system are evaluated based on 1) maximum feasible wall heights, 2) required clearances and easements, 3) their ability to minimize groundwater seepage, 3) their ability to minimize traffic and noise impacts, and 4) their ability to reduce the total project timeline. Those wall systems satisfying the minimum requirements for each of these evaluations were recommended for either Zone 1 or Zone 2.

The installation procedures to be used for a particular wall or invert system vary in terms of impacts to 1) the construction zone, 2) groundwater and soil handling, and 3) construction scheduling, all of which affect the applicability of the system to the project.

The functional performance of each structural system must be analyzed in order to prove its applicability to this project. Important considerations include whether the wall system is proven for lateral force resistance or simply a means of providing a vertical or near vertical face of soil. In addition the existence of standardized procedures, details, and testing as a means for quality control and predictable behavior of the system is beneficial for all of the evaluated systems.

In addition to the previous criteria, the favored construction methodologies must provide for readily obtainable, transportable, and easily managed materials and equipment.

4 Global Methods

Global methods are defined as those structural systems that are appropriate for any location within the project, whether below or above the groundwater table. The proposed rail alignment dips below the design groundwater table in the region between Vine and Washington Streets and continues to a maximum depth near Evans Street. Approximately 1,300 feet east of Evans Street the bottom of the proposed trench elevates above the groundwater table. Throughout this alignment, the wall systems would be required to retain soil, stop groundwater infiltration and resist additional lateral loads due to bridges, parking lots, buildings, rail traffic, and other design surcharges.

Whereas, the invert systems would be required to stop groundwater infiltration, resist vertical hydrostatic pressure, and support live rail/maintenance traffic. By inspection, due to deep cuts, large surcharge loads, and hydrostatic pressures from groundwater, the walls used in Zone 2 are applicable to Zone 1. However, the proposed invert systems are only required in Zone 2. The wall and invert construction methods analyzed for global applications are:

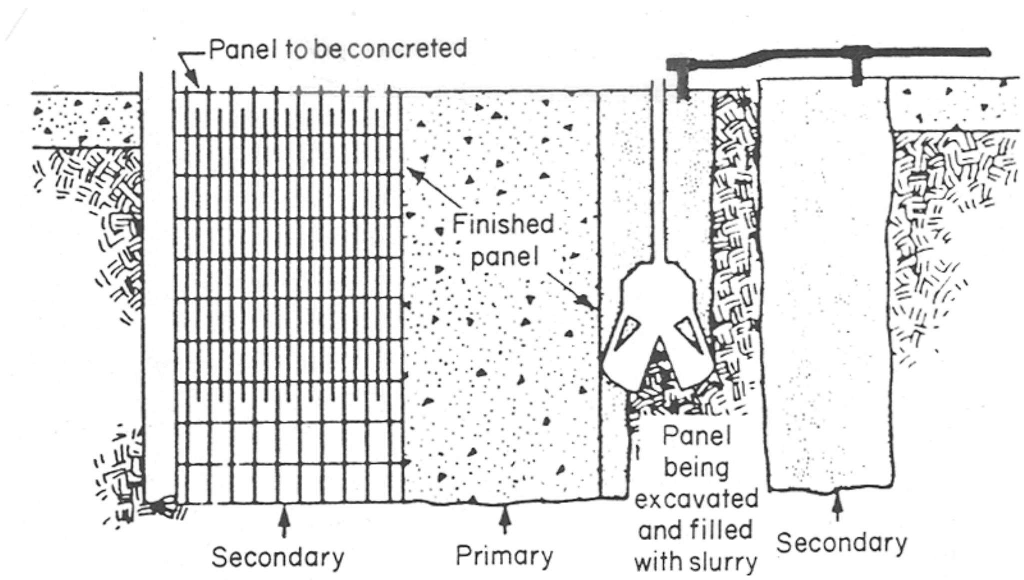
- Slurry-diaphragm walls
- Jet Grouting (walls and invert)
- Permeation Grouting (walls and invert)
- Secant/Tangent Pile Walls
- Cast-in-Place Concrete Invert

In the analysis that follows, the criteria used to evaluate each of the above wall and invert systems include 1) applicability to the City of Reno's subsurface conditions, 2) stability of wall construction to support heavy rail loads, 3) ability to stop groundwater infiltration into the finished trench section, 4) ability to support bridge structures [walls only], 5) rate and duration of construction, 6) construction effects on noise, traffic, and disruption of property access, 7) right-of-way impact [walls only], 8) aesthetic appeal [walls only], 9) cost, 10) history of successful application, and 11) overall applicability to the specific project site. Based on these criteria, recommendations have been made regarding applicability of each of the wall systems to the Reno Railroad Corridor.

4.1 Slurry-Diaphragm Walls (Zone 1 or 2)

4.1.1 Methodology

Slurry-Diaphragm wall construction is a methodology considered for the wall systems of the Reno Railroad Corridor project. These walls comprise continuous wall panels consisting of steel reinforced cast-in-place concrete, precast concrete panels, or steel sheet piles (listed in order of frequency of use) placed within an excavated trench temporarily filled with slurry. The slurry, a mixture of bentonite (sodium montmorillonite) and water, is used to prevent caving and water intrusion as the excavation for the wall panel proceeds downward from the ground surface. The excavated trench for a typical wall panel is generally 2 feet to 3 feet in width and 8 feet to 24 feet in length. The lateral dimension of the trench is maintained by utilizing cast-at-grade guide walls that are typically 3 feet deep and 2- to 3-feet wide. In appropriate conditions, a slurry trench panel can be excavated to depths well in excess of 100 feet. The panels are excavated such that every other panel is excavated and backfilled consecutively. During the next phase of excavation, the alternate panels are completed and so joined with the first group to create a uniform reinforced concrete wall (See Figure 4-1). Typical joint details include casting steel I-beams or pipes at the ends of each panel. In the case of the I-beam, the member is left in-place, as the adjacent panel is poured. However, in the case of the pipe end joint detail, the pipe is withdrawn as the adjacent panel is poured.



Ryan, 1987

Figure 4-1 Slurry-diaphragm wall Construction

Usually, hydraulically or cable operated clamshell buckets are used to excavate the slurry filled trench. Impact tools, chisels and minor blasting are methods employed to remove obstructions and solid bedrock.

In vibration-sensitive areas, or for very deep walls, clamshells and chisels can be replaced with milling (hydromill) equipment. The hydromill is an excavation device consisting of two milling heads rotating in opposite directions about axes perpendicular to the trench. The hydromill extracts the loosened dirt as it digs, replacing it with slurry to keep the walls from collapsing until the concrete is tremied (a technique for depositing and consolidating concrete underwater from the bottom upward). Since concrete is heavier than slurry, slurry rises to the top of the hole as the concrete is placed. Once at the top of the hole, the slurry is piped to a tank for cleaning and reuse.

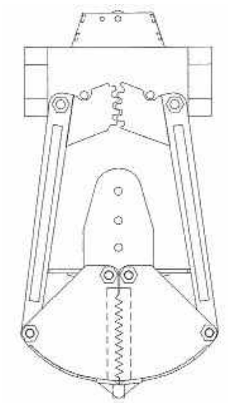


Figure 4-2

A variation of the slurry concept, restricted by depth and geology, is the continuous slurry-diaphragm wall. Highly specialized equipment excavates and places concrete in one operation eliminating the need for slurry to stabilize the trench.

Another variation of the slurry-diaphragm wall is the Soldier Pile Tremie Concrete (SPTC) wall. In this technique, steel soldier piles are placed in drilled holes that are then backfilled with lean concrete. The space between the soldier piles is then excavated with a trenching clam. Slurry is used to stabilize the trench until the trench is filled with tremied concrete. The steel soldier piles provide wall reinforcing instead of a reinforcing cage.

Figure 4-3 Reinforcement Placement

4.1.2 Applicability to Soil Conditions

Slurry-diaphragm walls using conventional slurry techniques are suitable for construction in all soil types. Experience indicates that progress through very dense soils and/or rock is possible. However breaking large obstructions is more time consuming and costly compared to construction in uniform soils without these large boulders. Clamshell equipment can often deal more effectively with boulders than larger diameter drills used in other techniques, thereby increasing the production rate of slurry-diaphragm walls when compared to drilled options. To overcome the challenges of obstructions hammers and chisels have successfully been used to fracture large boulders, piling and other debris to allow for removal. In addition, large boulders have also been fractured through the use of calculated placement and sizing of explosive charges.

When charges are employed, the contractor typically determines the size of the boulder through perforation with a small diameter drilling rig. The charge is then sized for the boulder and placed inside the drilled hole. After igniting the charge, the boulder pieces can be excavated with a clamshell. Although effective, this technique is typically not necessary for obstruction removal.

After removal of large obstructions, the slurry temporarily fills the void. As the panel is cast, the slurry is replaced by concrete, leaving a "bulge" or overbreak in the wall segment. It will be necessary to trim the overbreak at these locations to ensure a minimum of clear trench width of 54 feet.

Regions of open-work gravel and cobbles may affect slurry retention and wall stability. The soils in the City of Reno vicinity have void spaces and are cohesionless, making them susceptible to rapid slurry loss and trench sloughing. Enough slurry would have to be pumped into the trench to fill the voids or the slurry mix would need to be adjusted accordingly for stabilization of the adjacent trench walls at the Reno site. Close monitoring, specific geotechnical tests, and specially engineered slurry mixes may also be used to minimize the risk of rapid slurry loss.

The SPTC wall technique is also suitable for all soil types although somewhat slower progress may result from the need to drill large diameter holes. However, the project-specific soil conditions are not considered suitable for the hydromill or the continuous slurry-diaphragm wall device due to the presence of hard and large cobbles and boulders.

4.1.3 Stability of Wall Construction

Special geotechnical consideration is required for slurry-diaphragm walls. In the proposed methodology, excavations required for slurry-diaphragm construction are held open using bentonite slurry until they can be backfilled with structural concrete. The outward pressure provided by the slurry, on the sides of the excavation, resists the lateral forces that cause caving. However, a major concern of applying this technique to the excavation in the Reno Railroad Corridor is the stability of this system when employed adjacent to live heavy rail.

During the construction of this proposed trench system, Union Pacific will be operating live rail. This operation may be within 16 feet of the slurry filled excavation. Examination of this operation was performed to determine the susceptibility of the trench to caving and the magnitude of vertical displacements under the UPRR track. As shown in the detailed analysis of the slurry filled trench (Appendix D) the lateral and vertical displacements were determined to be 1/3-inch and 1/4-inch, respectively. Through consultation with Union Pacific Railroad, it was determined that this vertical displacement is acceptable for shoofly operation. In addition, the lateral displacement of ¼-inch is negligible in the analysis of global trench stability.

4.1.4 Groundwater Control

The coefficient of permeability is adjusted to account for placed concrete quality using top down construction techniques. Based on research (Dr. Donald Bruce, see Appendix A), permeability rates for a slurry-diaphragm wall system are on the order of $1x10^{-7}$cm/sec, compared to $1x10^{-9}$ cm/sec in high quality concrete.

Permeability values may be reduced by using high cement-to-water ratios (maintaining a proper balance for adequate freeze-thaw performance), concrete admixtures or high-quality precast concrete panels. A conservative evaluation was made using infiltration rates with permeability of $1x10^{-7}$cm/sec and a mean wall thickness of 3 feet. With these values, the completed walls are estimated to seep approximately $1.40x10^{-3}$ gal/ft^2/day. Comparing the seepage rate to the evapotranspiration rate in the City of Reno vicinity ($9.00x10^{-2}$ gal/ft^2/day), these walls are virtually watertight. Except on cold, humid days, these walls would not even feel moist to the touch. One of the largest advantages to the slurry-diaphragm wall technique is the small number of joints in the walls (compared to secant/tangent piles and jet grouting) - the most probable location for water infiltration.

Examination of the infiltration rates at the joints in this construction technique is difficult. However, through construction experience, it is estimated that the vertical joints may leak approximately 0.5 gal/min. per 100 feet of wall[2] during the initial stages of construction. Once these leaks are exposed, corrective work may be done to reduce these trouble spots to patches of moisture.

4.1.5 Abutment Related Issues

Large load carrying capabilities, compatibility with secondary lateral support systems, and minor resulting construction schedule impacts suggest the use of slurry-diaphragm walls at bridge abutments.

Typical high-cantilever walls used in bridge construction have a wall thickness approximately the same as the slurry-diaphragm walls used in this option. Therefore, design of slurry-diaphragm wall sections for bridge loads is relatively straightforward. Additionally, slurry-diaphragm walls are compatible with secondary lateral support systems such as grouted ground anchors or struts, which may be used to enhance their lateral performance. Furthermore, experience shows that if various wall types are determined to be necessary, detailing connections between slurry-diaphragm walls and other rigid retaining structures would be simple, because this wall technique eliminates the need to change wall types near bridge abutments.

Secondary systems have construction scheduling and right-of-way impacts associated with installation of tieback anchors and struts, and are discussed in detail later in this report.

4.1.6 Duration of Construction

Slurry-diaphragm walls are commonly used and standardized procedures, equipment, details, and testing are available. The top-down nature of this technique helps reduce the construction time frame by eliminating the need to de-water and excavate before construction begins. Since dewatering is not required to construct slurry-diaphragm walls below the groundwater table, construction schedules for Zones 1 and Zone 2 are presented.

[2] George J. Tamaro, Mueser Rutledge Consulting Engineers, New York, NY

Based on a single crew working on the wall system, approximately 800 square feet of wall surface area could be constructed in a single day[3]. For production, an accelerated schedule would be possible if a single crew were working on the east heading simultaneously to a single crew working on the west heading. These production rates exclude construction of secondary systems, chance encounters with large boulders or mass excavation of the trench section. Additional time should be included in final design to adjust these numbers to be compatible with selected construction methodology and equipment.

4.1.7 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Corridor.

Two major factors affect noise and traffic impacts from construction of wall systems in the Reno Corridor: equipment use and duration of construction.

The following is a brief list of the major construction equipment used to construct slurry-diaphragm walls:

- Clamshell
- Front loader
- Soil hauling trucks
- Slurry pump
- Concrete delivery trucks (Optional)
- Concrete pumping crane
- Reinforcement cage lifting crane
- Chisel for rock fragmenting
- Materials delivery trucks
- Pipelines
- Slurry Mixing and separation equipment

4.1.7.1 Noise Impacts

The vehicles listed above are common in the majority of all excavation construction sites, with the exception of the chisel or other devises required to fracture boulders. The fracturing operation would be conducted under bentonite slurry, which would dramatically reduce noise generated during use of this technique. These additional construction vehicles as listed above, although not quiet, introduce only minor levels of noise. This noise is above that of average daily traffic in the downtown Reno vicinity. Additionally, the combination of all the construction equipment required for this technique create less noise than is produced by train whistles in the current rail configuration.

4.1.7.2 Traffic Impacts

It is anticipated that slurry-diaphragm construction for the walls may be accomplished within the trench right-of-way while the trains are operating on the mainline tracks. This capability reduces actual traffic impacts by allowing the trains to operate on the main alignment for an extended period of time. Requiring only limited space to batch slurry and structural concrete and handle spoils.

[3]Michael Pagano, Area Manager, TREVI ICOS Corporation, Morgan Hill, CA

Figure 4-4 Light Rail Operations in New Jersey

Slurry-diaphragm construction requires special handling of slurry and soil, staging areas for construction equipment and supplies, and clear travel ways for crane operation.

Excavated soil must be drained in a 200 ft^2 pit, before transporting it to a treatment or landfill facility. There would be a minimum of two pits required, alternating soil draining and hauling. Additional handling is also required for the bentonite slurry. Recycling of the bentonite slurry requires a staging area of approximately 60 feet square.

Staging locations for cranes, concrete, steel reinforcement, and precast elements require an area of approximately 200 ft^2. Clear access is necessary, since this staging area would supply the wall production team with the necessary construction elements.

Figure 4-5 Spoils and staging

Based on the total required staging area (280 ft^2) the need to keep the slurry and concrete batching within 1,000 feet of the construction area, and the desire to keep trains in operation on the main tracks during wall construction, it would be necessary to construct staging areas outside the trench limits.

By constructing staging locations outside the trench limits, the contractor would be required to use adjacent parking facilities, warehouse storage yards, or adjacent city streets, thereby affecting city traffic by reducing parking, increasing local traffic congestion, or by the use of detours.

4.1.8 Right-Of-Way Impact (Wall Systems Only)

Slurry-diaphragm walls would require secondary lateral support systems in locations of large surcharges, bridge loads, high soil loading, and high groundwater. These added forces are resisted by these secondary systems to minimize the structural section of the wall. Some secondary systems have right-of-way impacts, most notably, grouted ground anchors. These wall systems are compatible with struts or grouted ground anchors or a combination of both.

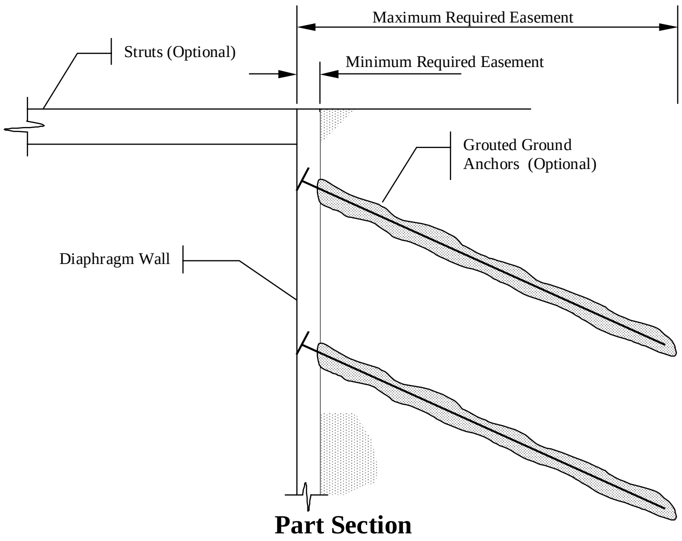


Figure 4-6 ROW Impacts

At bridge locations, it is reasonable to use tieback anchors, since the right-of-way is directly under city streets. Additionally, sections of the alignment where the right-of-way is greater than 50 feet on either side of the trench and where an unobstructed work space is required are well suited for grouted ground anchors. Other locations may be best suited for struts that brace the wall systems against each other, passing laterally across the top of the trench.

The structural section of the slurry-diaphragm wall is approximately 3 feet thick. Therefore, in regions that do not require additional lateral support, the maintenance easement is only the width of the wall.

4.1.9 Aesthetics (Wall Systems)

After trench excavation, slurry-diaphragm walls would have natural relief on the surface of the wall. The relief on the wall is caused by the cobbles and large granules found in the native soils. The final surface may have a thin coating of bentonite and may require some finishing to remove overbreak, larger boulders or other inclusions. After finishing, due to the cobbles and boulders in the Reno vicinity, it is anticipated that the wall surface will not necessarily be neat nor clean, unless precast panels are employed. However this is not necessarily detracting and options for aesthetic treatments applied to slurry-diaphragm walls are varied. These options include façades,

Figure 4-7 Untreated Basement Wall

shot-creted facing walls, and formliners.

The use of façades involves hanging panels from dowels embedded in the slurry-diaphragm wall. Façades are available in many types of materials, colors, and textures. However, in regions of extreme temperature change, like the City of Reno, these treatments must be designed for freeze/thaw conditions. The addition of a façade would add approximately $15/ft^2 and 20% to the construction timeframe. Façades are the most complex aesthetic treatment available.

Shot-crete is a low slump, highly plastic concrete sprayed over a mesh of reinforcing bars that is attached to the slurry-diaphragm wall with reinforcing steel dowels. A shot-creted wall has similar advantages to a cast-in-place reinforced concrete retaining wall. The finishing of these facing walls can be adjusted to include color, texture, and feature lines. The addition of a facing wall adds approximately $3/ft^2 and 5% to the construction schedule.

Another aesthetic treatment involves the use of precast panels. Precast panels can easily be modified for aesthetic applications. These panels, unlike façades, can be designed to be the primary element resisting the loading imposed on the wall. Additionally, these panels can be cast off-site with features including color, texture, or feature lines. The addition of precast panels adds approximately $10/ft^2 and may decrease the time required for construction by eliminating the need for curing and reducing installation requirements.

Various other options are possible for aesthetic treatment of the finished surface. One example would be the inclusion of plywood with or without formliners. Inclusion of plywood (with a bond breaker applied), would provide a clean plane to for partial concrete removal, leaving the appearance of windows, or other artistic features. The addition of plywood and formliners would add approximately $4/ft^2 and $5/ft^2 to the total project. Furthermore, if formliners are used, it is necessary to include the added cost of the plywood, yielding a total cost of approximately $9/ft^2 for this solution.

Although slurry-diaphragm walls are easily modified for aesthetics, it is not recommended to apply either façades or facing walls. These applications would not allow for simple wall maintenance and may pose other risks associated with freeze-thaw conditions. Additionally, the facing may hide wall leak sources. These sources, if present, can be fixed by grouting behind the wall. Not visible, leaks are difficult, or impossible, to repair. Therefore, if slurry-diaphragm walls are chosen and aesthetic treatments are necessary, it is recommended to contemplate the use of precast panels.

4.1.10 Conceptual Calculations

Slurry-diaphragm walls rely on the capacity of the reinforced concrete wall to resist the bending and shear imposed by lateral loads from soil, surcharges and groundwater. The design of these walls is similar to sheetpiling and traditional reinforced concrete cantilever walls. The design procedure is outlined below:

Determine lateral loading condition
A determination of lateral loads is accomplished by using Rankine's method to find active earth pressure on a structure, combined with analysis of surcharge and groundwater loads. The surcharge loads applied to the wall system included loads from surrounding structures, parking lots and Cooper E-80 rail loading (where applicable).

Internal Force Determination
The walls are analyzed as simply supported beams in two directions. The first direction is vertical. With these forces, the design for the vertical steel along the front and rear face can be completed. Then, horizontal analysis determines the forces for horizontal reinforcement design.

Design
Using American Concrete Institute's code regarding reinforced concrete design (ACI 318-99), the design proceeds for bending and shear criteria. The front face of the wall is designed for the moment applied in the final wall configuration, while the back face steel is designed for the temporary construction condition of being an unsupported cantilever element. In addition, the horizontal steel in the front face is designed to resist bowing effect of the wall between the strutting supports at the top of the wall. After the wall configuration is established for bending criteria, the wall section is checked against code shear requirements.

Findings
Using the wall configuration shown below and a compressive strength of concrete of 2,500 psi, the wall section is required to be a minimum of 2 feet thick (for constructibility) and a minimum of 3 feet thick for design heights of 10 to 35 feet (3 feet thick in Zone 2), measured from the top to the bottom of the trench. The required steel reinforcement is approximately 1% by volume. Typically, cantilever walls without footings must be extended a great distance below the final excavation to provide enough passive soil resistance to counteract overturning forces, However, with a jet grouted base, (Section 4.2) this lengthy extension is minimized by the large passive pressure capacity of cemented soil. Additionally, in examining the unbraced condition during construction, it was found that excavation could proceed to a depth of approximately 10 feet without bracing installation.

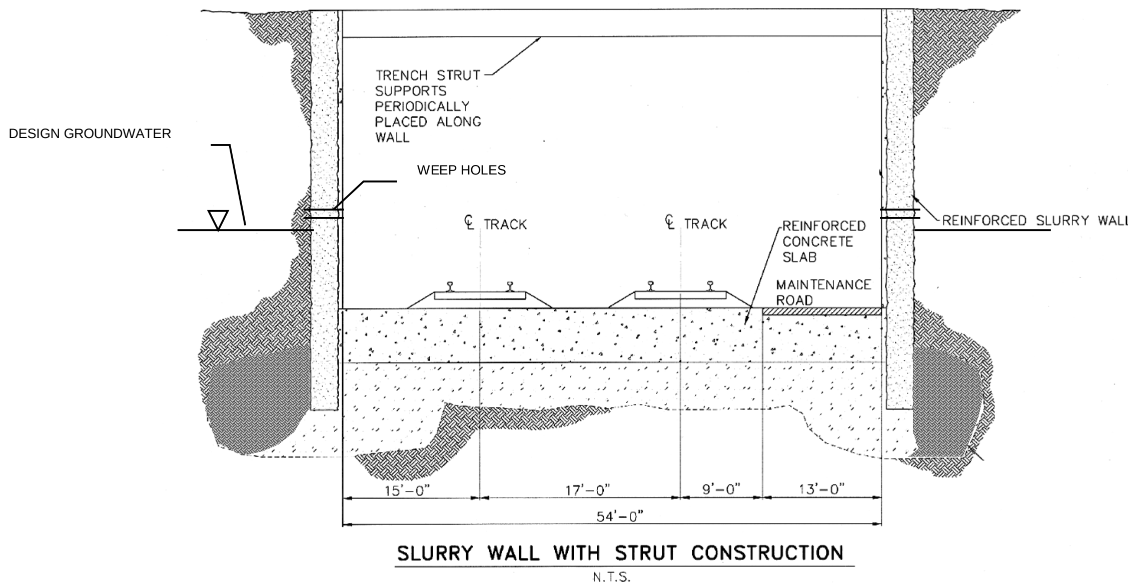


Figure 4-8

Another secondary support system is available for use with the slurry-diaphragm wall concept. Grouted ground anchors can be installed through the exposed face of the wall during trench exavation. The effect of grouted ground anchors is similar to that of struts, as it allows for a reduction in the structural section of the wall.

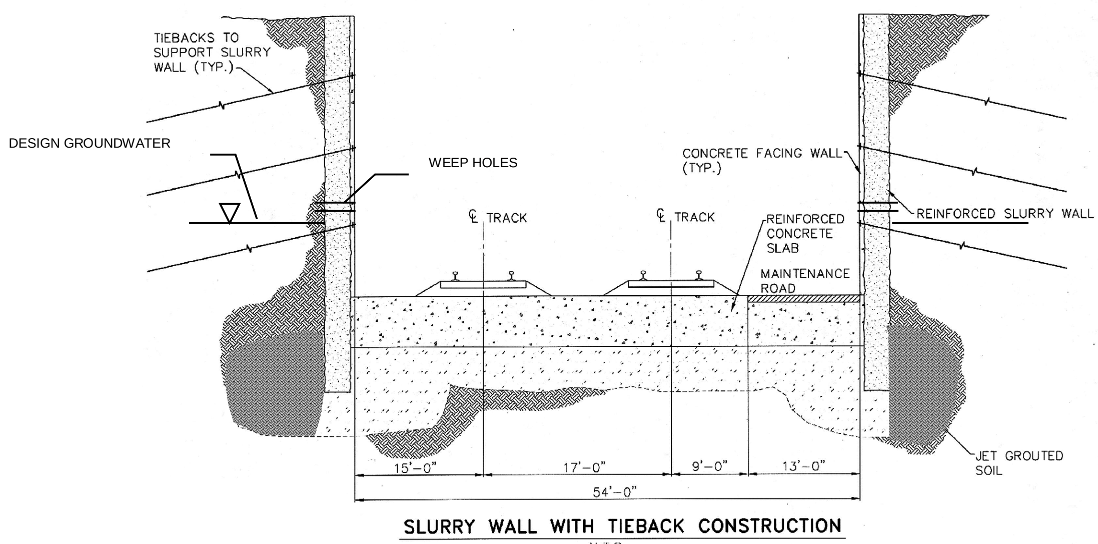


Figure 4-9

4.1.11 Cost[4]

Evaluation of the approximate costs was based on informal discussions with specialty contractors experienced in slurry-diaphragm wall construction. For

[4] Mario Mauro, Trevi-Icos, Boston, MA

preliminary purposes, an estimated cost of $\$55/ft^2$ for typical construction is appropriate. However, for the Reno Railroad Corridor project, it is estimated that the difficult geology would add approximately $\$15/ft^2$ to the final project costs (total wall construction costs $\$70/ft^2$). Also, the addition of grouted ground anchors would increase the cost another $\$15/ft^2$ ($\$85\ ft^2$ wall cost with anchors). Since aesthetic treatment is optional for these walls, the additional costs attributable to aesthetic treatments are not presented here. For more information on aesthetic impacts, see the Aesthetics section of this report.

4.1.12 History of Successful Application

Below is a list of successful projects with construction and performance criteria similar to those required for the Reno Railroad Corridor:

World Trade Center, New York, NY

Constructed in 1967, this project encompasses 16-acres to form the basement of the World Trade Center (4 city blocks wide by 2 city blocks long). The 3-foot thick walls extend below grade to a depth of approximately 70 feet. Panels were constructed in 23-foot lengths in plan. The project took approximately 1½ years to complete the wall systems and release the temporary tiebacks. Groundwater was encountered within 5 feet of the original surface during excavation. This wall sustained the collapse of three supporting floor diaphragms during the bombing of the structure in 1996, spanning 30 feet vertically between supports.

The most difficult aspects of this project included construction in the various types of soil, including glacial till, man-made fill with debris, quartz, sand, silt, and finally keyed into bedrock.

Owner

New York and New Jersey Port Authority

Design Firm

Mueser Rutledge Consulting Engineers, New York, NY

Financial Square, New York, NY

Constructed in the mid-1980's, this 30-inch thick basement was constructed to a depth of 60 feet and covers approximately 2 square city block. Since this project is right on the water, there is a total of 60 feet of hydraulic head on the wall. This wall was constructed in 21- to 22-foot wide panels (measured in plan).

Owner

Merrill Lynch and American Express

Design Firm

Mueser Rutledge Consulting Engineers, New York, NY

Transitway "Court House" Station, South Boston, Massachusetts (Contract EO2CN14)

The project involved the construction of 167,000 ft^2 of structural slurry-diaphragm wall.

Owner

Massachusetts Bay Transportation Authority

General Contractor

J. F. White Contracting Company

Specialty Contractor

TreviICOS Corporation

Fort Point Channel, Boston, Massachusetts

Description

The project involved the construction of 68,000 ft^2 post-tensioned slurry-diaphragm wall.

Owner

Massachusetts Highway Department

General Contractor

Cashman/Perinin/Kiewit/Atkinson

Specialty Contractor

TreviICOS Corporation

Four Seasons Hotel and Tower, San Francisco, California

Site soils consisted of 15 feet of miscellaneous fill and old building debris overlaying 20 feet of loose sand. Beneath this was 60 feet of very dense sand, with very stiff "Old Bay Clay" encountered at approximately 95 feet below street level. Static groundwater was present at 30 feet. The 36-inch thick slurry-diaphragm wall was designed to follow the building perimeter, penetrating the dense sand to key a minimum of 10 feet into the underlying clay stratum.

Wall construction was performed by two crawler cranes operating cable-hung clam buckets. Panels, varying in length from 14 to 26 feet, were excavated and concreted in an alternating sequence to allow the concrete to set-up before adjacent work began. During the excavation process, each panel was continuously filled with pre-mixed bentonite/water slurry to provide ground support for each panel. When excavation reached design depth, a full-length reinforcing cage, fabricated on site, was lowered into place through the bentonite slurry. The panel was then tremie-filled with 5,000 psi concrete, displacing the slurry to a nearby holding tank. A total of 57 interconnecting panels were constructed to depths varying between 105 and 125 feet below street level to encompass the site perimeter (1060 linear feet). In addition to functioning as temporary excavation support and final basement construction, the completed wall also served as a permanent groundwater cut-off, eliminating the need for long-term construction dewatering. To date, this is the largest and deepest slurry-diaphragm wall ever constructed for a building project in San Francisco. Case completed the installation on schedule and produced an exposed wall of exceptional quality

Owner

Millenium Partners, New York, New York

Construction Manager

Bovis Construction Corporation, San Francisco, California

Specialty Contractor

Case Foundation

4.1.13 Advantages and Disadvantages of Slurry-diaphragm walls

Advantages:

- Properly constructed walls are relatively impermeable.
- Dewatering during construction not required.
- Can be constructed in all types of soil.
- Rigid walls allow spanning farther between bracing supports than other conventional wall systems.
- Significantly less lateral ground movements than more flexible wall systems.
- Can be used as permanent structural walls to support lateral and vertical loads, thus reducing or eliminating the need for secondary wall construction.
- Batching and fabricating on site reduce traffic impacts on local roads.
- Adaptable to bridge abutments.
- Specialty contractor.

Disadvantages:

- Higher cost in comparison to some other wall systems.
- Requires large staging areas.

- Disposal of slurry and slurry contaminated soil required.
- Crane operation requires headroom.

4.1.14 Application

4.1.14.1 Area of Use

Dewatering is not required for successful construction of slurry-diaphragm walls. Therefore, these walls systems are applicable both above and below the groundwater table. Slurry-diaphragm walls are compatible with secondary lateral bracing systems, can be successfully constructed in the native City of Reno soils, and are positive groundwater barriers. Therefore, these walls are excellent candidates for either Zone 1 or Zone 2. However, the added costs associated with this construction technique makes it less attractive in Zone 1 than conventional cast-in-place reinforced concrete cantilever retaining structures or other systems.

4.1.14.2 Installation Procedures

Major elements of the construction procedure, outlined below, indicate that slurry-diaphragm walls can be successfully constructed in difficult soil conditions. Additionally, these steps indicate the potential difficulties associated with top-down construction and include the procedure for a single panel. Each panel is approximately 15 to 20 feet in length (measured along the alignment).

1. Trench Excavation-
   The trench is excavated using clamshells. Upon striking large obstructions, large hammers (50 tons or more), chisels or minor blasting are employed to pulverize the boulders into pieces small enough to extract with clamshells. It may be possible to transfer UPRR service to the south rail during north wall construction and vice versa. The shoofly would not be required until the mass trench excavation begins.
2. Slurry Placement-
   In conjunction with the trench excavation, the bentonite slurry is added to stabilize the vertical walls from sloughing. This dense liquid can be pumped from the supply staging area several thousand feet away.
3. Reinforcement Placement-
   Large, pre-tied, reinforcement cages or steel piles are inserted in the bentonite slurry and positioned within the middle of the trench.
4. End Stop Placement-
   A construction joint with a shear key detail is inserted into the end of the panel (end stop).
5. Tremie Concrete Placement-
   After accurate placement of the reinforcement is accomplished tremie concrete is placed into the trench. The concrete is heavier than the bentonite slurry mix. Replaced by tremie concrete, the bentonite slurry is pumped from the top of the trench, cleaned, and recycled.

6. End Stop Extraction-
   As the concrete is hardening, the end stops are "tugged" and extracted soon after. After the extraction, the next panel is started.

If the precast panels are used, the panel is inserted in the bentonite slurry and joined to the adjacent segment with an interlocking shear key detail. Then the slurry is pumped out, and the next panel is prepared.

The top-down procedure includes difficulties with extraction of large obstructions and maintenance of the trench wall. These difficulties have been successfully surmounted in similar projects around the world. Although this construction method is anticipated to be difficult in localized areas, the procedures are possible throughout the Reno Railroad Corridor.

4.1.14.3 Functional Performance

Concrete slurry-diaphragm walls are typically the stiffest excavation support system available. Consequently, they are generally utilized in areas that require minimizing adjacent ground movements. In addition, properly installed walls are relatively impermeable. With sufficient penetration below the intended base of the excavation, a slurry-diaphragm wall can provide an effective seepage barrier that minimizes infiltration of groundwater into the trench.

These wall systems are used effectively and efficiently for similar applications worldwide and are anticipated to function extremely well in either Zone 1 or Zone 2.

4.1.14.4 Materials and Equipment Availability

Skilled labor is essential to effective wall construction, so a specialty contractor would be required. By using specialists for this construction technique, scheduling risk and project overruns are reduced. A specialty contractor also provides the equipment needed and transports it to the project location in the City of Reno. Additionally, the construction materials, namely concrete, bentonite slurry, and steel reinforcement, are common and are available within the City of Reno vicinity.

Based on a specialty contractor supplying equipment, labor, and commonly used construction materials, the construction of slurry-diaphragm walls would be feasible and practical for the Reno Railroad Corridor.

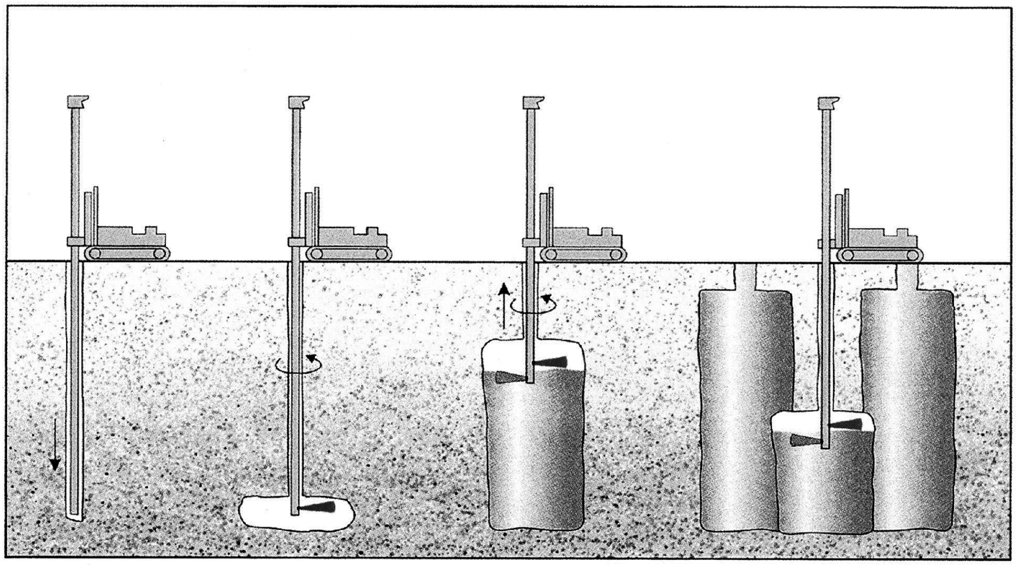
4.1.14.5 Conclusions

After an examination of the applicability of slurry-diaphragm walls to the Reno Railroad Corridor project, it was concluded that slurry-diaphragm walls are appropriate to use, are possible to build, pose no fatal flaw, and would function as required in Zone 1 or Zone 2.

4.2 Jet Grouting (Zone 1 or 2)

<u>4.2.1 Methodology</u>

Jet grouting is a ground treatment system used to create in-situ, cemented soil formations. This methodology is considered for the wall systems and a separate invert system of the Reno Railroad Corridor project. The method uses pressurized fluids to segregate and remove some of the soil particles then mix and replace them with a soil/cement mixture that has high strength and low permeability.



JET GROUTING PROCESS

Figure 4-10

Rotating high-pressure fluid jets, employed while withdrawing grouting rods from a predrilled boring, form cylindrical columns. Jet grouted walls or inverts are constructed by overlapping these cylindrical columns. The typical construction method is shown in Figure 4-10.

There are three generic jet grouting systems: single, double, and triple fluid systems. A single fluid system uses only a grout jet to simultaneously erode and mix the soil with the cementing agent. The single fluid system involves only partial replacement of the soil and produces the smallest cemented soil column diameter(2- to 3-feet).

The double fluid system uses a compressed air envelope around the grout jet. The compressed air improves the cutting ability of the grout jet for segregation and mixing, and creates a larger column diameter than the single fluid system (3- to 6-feet). However, the strength of the cemented soil mass constructed by the double fluid system may be reduced due to entrapped air. The system also produces more spoil than the single fluid system.

For the triple fluid system, fluids are emitted from two levels in the drill rod. An upper jet of water and air is used to excavate the soil, which is mixed with or replaced by jet grout emitted from a lower port. This system provides the largest diameter column (up to 12 feet) but also results in the largest amount of spoil and requires the largest quantity of cement. Since the water and air are not directly mixed with the grout, the air content does not affect the strength of the cemented soil.

For structural jet grouted walls, reinforcement may be placed in the drilled holes to increase the bending capacity. The installed reinforcement may be steel pipes (up to 8 inches in diameter), large steel reinforcing bars, rolled steel sections (up to 8 inches in depth), or the drill shaft used for grouting.

Contrary to wall systems, jet grouted inverts are not reinforced. To properly install reinforcement to resist internal forces would require the reinforcement to be placed horizontally. Since this system is constructed in a top-down fashion, access to the jet grouted mass is only available through the drilled holes, thereby eliminating the possibility of installing reinforcement in a beneficial orientation.

4.2.2 Stability of Wall Construction

A major concern of applying this technique to the construction of walls in the Reno Railroad Corridor is the stability of this system when employed adjacent to live heavy rail. However, with jet grouting, this concern is minimized by the technique itself.

Jet grouting uses pressurized fluids to segregate and remove some of the soil particles then mix and replace them with a soil/cement mixture. Unlike other open excavation methods, replacement of soil is almost instantaneous and localized, thereby eliminating the concern of producing an unstable soil mass that may collapse during construction. The potential for collapse with this method is limited to the diameter of the uncured grouted cylinder, approximately 4- to 6-feet. Therefore, this method offers stability benefits over open-excavation techniques.

4.2.3 Applicability to Soil Conditions

Jet grouting can be used for a wide range of soil types, but special care is required when dealing with very stiff and/or highly plastic clays. Local obstructions, such as boulders, can often be bypassed or encapsulated into the jet grouted soil mass. Predrilling and supplemental probes would be required to satisfactorily grout around boulder obstructions and achieve a reasonably effective seal, especially below the water table. Using supplementary probes or double passes, inadequately sealed joints from the initial jet grouting can be plugged, minimizing groundwater seepage. Additional methods to repair such inadequate seals could include localized permeation grouting.

Variations in soil fines content, gradation, and density are likely to result in irregularities in the radius of the jet grouted columns, increasing subsequent excavation difficulties where required to expose the wall or invert systems.

4.2.4 Groundwater Control

WALL SYSTEMS

Overlapping jet grouted soil masses with a thickness of 5 feet and greater (walls) or 6 feet and greater (inverts) would produce a positive groundwater barrier for these systems. In addition, temporary dewatering is not required for jet grouting; therefore, this soil treatment system is applicable in Zone 2. Furthermore, the geological conditions are conducive to jet grouting in Zone 1. Hence, for wall construction, this method is a viable option in both zones. Breeches in wall and invert integrity during construction that produce localized infiltration may be patched on an as-needed basis using additional rows of jet grout or other grouting techniques. It is estimated that jet grouted columns with a maximum diameter of 4.5- to 5-feet are possible for construction in City of Reno subsurface conditions. With columns spaced so closely, joints between the columns are numerous. These joints provide locations of possible localized infiltration, producing an effect, in the walls, that resembles vertical blinds. For this reason, jet grouting, although a proven technique, contains more unknowns than other wall construction options with fewer joints. Due to the unknowns, the preliminary stages of design must include a contingency fund that provides for localized patching of the wall systems.

INVERT SYSTEMS

Examination of a jet grouted invert system in the Conceptual Calculations concludes that the grouted mass must be approximately twice the depth of the hydraulic head over its base. Therefore, the thickness of the jet grouted layer will vary from a minimum of 3 feet and a maximum of 10-feet 9-inches. When comparing this thickness to the permeability analysis, it is seen that a properly constructed jet grouted invert, in these conditions, can be made almost watertight.

However, jet grouted invert systems are installed by intersecting adjacent grouted columns (as shown below) creating more possibilities of localized breeches, when compared to a cast-in-place option. For these reasons, jet grouted inverts would only provide the necessary groundwater barrier for temporary conditions of constructing a permanent invert solution.

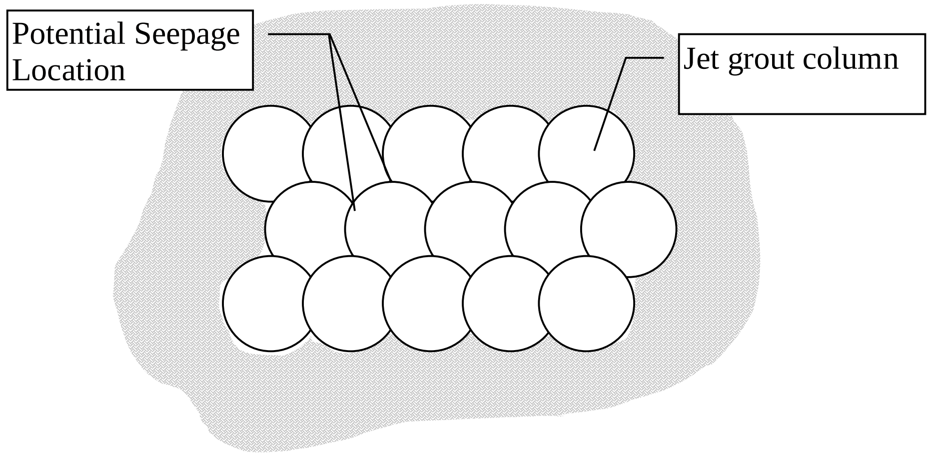


Figure 4-11 Partial Plan of Jet Grouted Columns

Jet grouting to temporarily reduce groundwater infiltration during construction of a different invert system is possible. The jet grout would allow for manageable extraction of trapped groundwater during construction. Due to the complex geology and limited geotechnical investigation, a test section for this technique would be essential to evaluate potential seepage and actual construction rates.

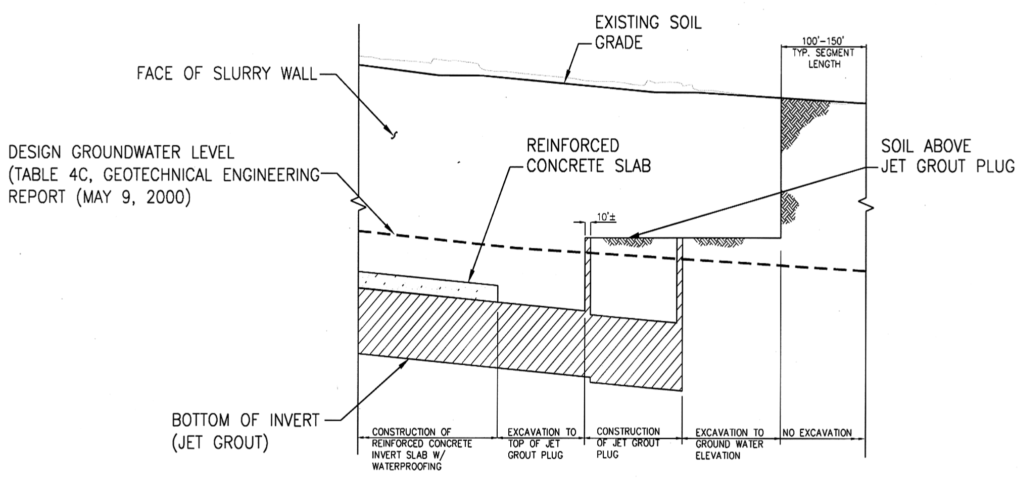


Figure 4-12 Longitudinal Section of Trench Depicting Segment Construction

Furthermore, construction of this technique for the invert system would probably be conducted in reaches of approximately 150- to 200-feet (as seen above). Each reach ending with a series of contiguous jet grouted soil columns extending up past the groundwater line. This segment would then be excavated and pump testing would be performed to ensure a proper seal. Remediation work may be required in the segments to correct excessive leakage. Successive segments could continue after proper sealing is achieved.

4.2.5 Abutment-Related Issues (Walls Only)

Jet grouted walls are typically unreinforced, however a simple modification to the installation procedure (leaving the shaft in the predrilled hole and inserting large steel pipe column sections -3 to 6 in. diameter - or steel reinforcing) has been a successful approach to increasing the flexural capacity (ability of the wall to resist bending) of jet grouted soil masses. The exterior surface, having a relatively high moisture content, would not weather well in freeze-thaw conditions. Localized spalling and degradation of the exposed jet grouted surface are likely. The brittle, but strong, nature of these walls precludes them from being used as substructure elements for the abutments. Use of another system, such as slurry-diaphragm walls, cantilever walls, or a deep abutment foundation would be required in these locations.

Ensuring watertightness and soil retention is challenging, given the dramatic differences in structural types between jet grouted walls and traditionally reinforced concrete high-cantilever abutments.

4.2.6 Duration of Construction

Jet grouting requires pre-drilling approximately 6- to 8-inch diameter holes (for insertion of the grouting tube through the soil stratum to the bottom of the column) and injecting the grout as the tube is withdrawn. After completion of the column, localized patching may be required, which necessitates additional drilling. The preliminary geotechnical investigation, using ODEX equipment, proved that each hole drilled with this specific equipment was a time consuming process. However, it is anticipated that the pre-drilled holes for this technique would utilize a down-the-hole hammer, which is quicker and less expensive than the ODEX equipment used for the geotechnical investigation. Though slower in the City of Reno vicinity than in other locations without cobbles and boulders, drilling 6- to 8-inch diameter holes is faster than drilling 1-meter diameter holes.

WALL SYSTEMS

Using the ODEX production data (135 ft/shift)[5], each hole spaced at approximately 6 feet on center, and an average of two rows of columns, the wall section in Zone 1 would require approximately 450 shifts. In Zone 2, the construction of jet grouting would require 636 shifts. The actual grouting process would be only a fraction of the drilling time.

INVERT SYSTEMS

Using the aforementioned production rates for the ODEX method and a spacing of approximately 6 feet on center, the invert would require approximately 1,043 shifts.

Based on demonstrated production rates, which are all well within the 24-month anticipated timeframe, jet grouting is feasible for both wall and invert construction throughout the project.

[5] Mark Doehring, Kleinfelder Inc.

4.2.7 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Railroad Corridor.

Two major factors affect noise and traffic impacts from construction of trench systems in the Reno Corridor, namely, equipment use and duration of construction.

The following is a brief list of the major construction equipment used to perform jet grouting:

- Drill Rig
- Jet Grouting Rig
- Grout pump
- Batching Plant
- Materials Delivery Trucks
- Spoils Hauling Truck

4.2.7.1 Noise Impacts

Noise impacts due to jet grouting, particularly down-the-hole air percussion drilling are moderate when compared to daily traffic and normal train operation. Based on noise impacts, this technique is viable throughout the trench alignment.

4.2.7.2 Traffic Impacts

WALL SYSTEMS

It is anticipated that jet grouting for the walls may be accomplished while the trains are operating on the mainline tracks. This capability reduces actual traffic impacts by allowing the trains to operate on the main alignment for an extended period of time. Requiring only limited space to batch grout and handle spoils, the staging areas for jet grouting are minimal and reduce the need to vacate nearby parking lots or city streets—one of the largest benefits of the jet grouting procedure.

INVERT SYSTEMS

It is anticipated that jet grouting for the invert may be accomplished within the trench right-of-way while the trains are operating on the shoofly. This increases actual traffic impacts by requiring the trains to operate on the shoofly during construction. Requiring only limited space to batch grout and handle spoils, the staging areas for jet grouting are minimal and reduce the need to vacate nearby parking lots or city streets—one of the largest benefits of the jet grouting procedure.

4.2.8 Right-Of-Way Impact (Wall Systems Only)

Jet grouted wall systems require secondary support systems and a wall section approximately double the size of reinforced concrete designs (cantilever walls, slurry-diaphragm walls, and secant/tangent pile walls).

Jet grouted soil walls would not be able to withstand the lateral forces produced by a 35-foot-high cantilevered wall. The maximum jet grouted cantilever capable of resisting the lateral loads is approximately 8 feet. Intermediate supports are required for jet grouted walls spaced at approximately 6 feet on center (measured vertically). Due to this requirement, vertical clearance cannot be obtained using struts only. Therefore, grouted ground anchors in combination with wales (horizontal members running parallel to the wall) are the best secondary support solution for jet grouted walls.

The use of grouted ground anchors in jet grouted walls introduces right-of-way concerns, since larger underground easements are required. Throughout the trench alignment, the shortest underground easement (in the areas of the trench ends) is expected to be 15 feet. Since the walls are shorter (average heights of 14.5 and 11.5 in these areas), the tieback anchor lengths required are within the proposed underground easements, as presented in the DEIS. In the taller wall sections, the underground easements are expected to range from 35 to 50 feet beyond the front face of the wall. The maximum length required for tieback anchors in these sections is proposed at approximately 50 feet. Since the thickness required for the jet grouted columns is less than the 50-foot permanent easement proposed in the DEIS, jet grouted walls do not pose adverse right-of-way impacts that would preclude them from being used in the Reno Railroad Corridor.

4.2.9 Aesthetics (Wall Systems)

Jet grouted walls are similar in appearance to traditional slurry-diaphragm walls (with a convex surface), having 4- to 6-inch relief and a cemented native soil surface. However, aesthetic treatment options for jet grouted walls are not as plentiful as those for slurry-diaphragm construction. The brittle, unreinforced walls (located in the freeze/thaw region of Reno) require special detailing and it is costlier to install aesthetic treatments such as a façade or facing wall. Therefore, in the regions where the aesthetic appearance of cemented native soil is unacceptable, the use of jet grouting would be discouraged.

4.2.10 Conceptual Calculations

WALL SYSTEMS

Jet grouted walls are typically designed as gravity wall systems, however they may be designed with secondary lateral support systems and/or reinforced with vertical members as discussed in sections 4.2.5 and 4.2.8. For this discussion, the preliminary calculations assumed gravity wall behavior.

Jet grouted walls must be large enough to resist overturning and provide enough base friction to resist sliding. For design purposes, it is assumed that these wall systems would produce unit weights of the jet grouted mass on the order of 140 pcf and a coefficient of static friction of 0.35. The wall thickness in Zone 1 is typically on the order of one-half the wall height. Due to hydrostatic pressure developed behind the walls in Zone 2, the required wall thickness is greater.

In Zone 1, the average wall heights are 14.5 and 11.5 feet for the eastern end and western end, respectfully. Construction of these walls would require 7.25- and 5.75-foot-thick walls, based on an average diameter of 4.5 to 5 feet for jet grouted columns requiring two rows of columns, as would be appropriate to City of Reno geology.

In Zone 2, the average wall height is 28.5 feet, requiring a wall thickness of approximately 15 feet. Based on the jet grouted column diameters expected in Reno, the construction of these walls would require 3 to 4 rows of grouted columns.

INVERT SYSTEMS

Examination of the jet grouted invert option required an analysis of two separate design constraints: 1) buoyancy limits and 2) practical construction criteria. Each of these constraints is described below.

Determine Buoyancy Limits

The critical design for buoyancy was calculated for the condition in which the invert was constructed and excavation was completed to the top of the jet grouted plug, thus eliminating any overburden. For this analysis, an iterative solution was used to vary the thickness of the invert plug and calculate the buoyancy and opposing forces.

As described in Groundwater Control, it is preferable to design a jet grouted invert system as a temporary construction method, allowing for construction of a cast-in-place alternative. Therefore, different factors of safety and design criteria will be applied to this system compared to the permanent solution. Specifically, the groundwater elevation used to determine the buoyant forces was taken as the highest measured groundwater level during the yearlong monitoring conducted by Kleinfelder. Moreover, the factor of safety was reduced to be consistent with temporary construction conditions, namely 1.2. Using this criterion, buoyant forces were determined.

The buoyancy force is created through a differential head (h) between the bottom of the invert plug and the highest observed groundwater elevation. The differential head was determined using the measured distance between the top of the invert plug (determined by layout geometry) and the highest observed groundwater table elevation (determined through site exploration) and adding the trial invert thickness. The buoyancy force was determined by multiplying the differential head by the unit weight of water, subsequently increasing the value by a 20 % safety factor. Counteracting the buoyant force is the unit weight of the invert plug.

Ignoring any slab bending performance, the unit weight of the invert plug was determined by multiplying the trial invert thickness by the assumed unit weight of the jet grouted mass (140 lb/ft^3)[6].

After determination of the opposing forces, they were compared to ensure the unit weight of the mass was greater than the design buoyant force.

These forces are extremely sensitive to fluctuations in groundwater elevations. Therefore, in final design, it is imperative that the highest probable groundwater elevation be included in these calculations. Alternatively, the elevation of the groundwater table that effects jet grout thickness may be held constant through

[6] Donald A. Bruce, PhD., GEOSYSTEMS, L.P., Venetia PA

the employment of weep holes or other pervious detailing at the highest acceptable groundwater elevation. In a natural event that elevates the groundwater table over this threshold value, the water would flood the trench rather than fail the invert and wall system.

Practical Construction Criteria

Application of practical construction criteria is difficult in preliminary engineering. Each construction team may approach this differently using different materials, equipment and methodology. Safety factors and site-specific geotechnical data are also a concern for developing practical construction criteria. Therefore, for the purposes of this study, it is assumed that the minimum practical thickness of jet grouting is 3 feet.

Results

Using the criteria, assumptions and procedures outlined above, minimum, maximum, and average jet grouted plug thickness were determined as summarized below:

Minimum Thickness: 3'-0"
Maximum Thickness: 10'-9"
Average Thickness: 7'-11"

Based on the values above and the plan and profiles shown in Appendix C, the total volume of treated soil for jet grouting is approximately 81,900 yd^3.

4.2.11 Cost

WALL SYSTEMS

Using the conceptual calculations developed for a gravity wall system, a costs analysis was conducted. The typical cost for jet grouting, including the drilling required, is approximately $180 per treated cubic yard[7]. For Zone 1, with a wall thickness of 7.25 feet and an average height of 14.5 feet, the cost per square foot of wall is $49. In Zone 2, with a wall thickness of 15-feet and a wall height of 28.5 feet, the cost per square foot of wall is $101/$ft^2$. Using these values and assuming application of jet grouting throughout the entire alignment, the weighted average cost for jet grouting is $80/$ft^2$.

INVERT SYSTEMS

Similar to the wall systems, the typical cost for jet grouting for the invert, including the required drilling, is approximately $180 per treated cubic yard. Examining sections of the trench in 200-foot lengths, the calculated volume for treated material in the invert plug is approximately 81,900 yd^3, yielding a cost of $14.7 million ($53/ft^2 - measured in plan).

4.2.12 History of Successful Application

Below is a list of successful projects with construction and performance criteria similar to those required for the Reno Railroad Corridor:

[7] Donald A. Bruce, PhD., GEOSYSTEMS, L.P., Venetia PA

Project:

Central Steam Plant, Spokane, Washington

Description:

Jet grouting technology was used to successfully construct a below grade 280-foot-long barrier wall to contain petroleum hydrocarbon contamination in downtown Spokane, Washington. The site geology consisted of three primary units: unconsolidated sand and gravel, silt, and bedrock. The sand/gravel unit varied in thickness from approximately 10 to 20 feet along the southern boundary of the site to approximately 50 feet near the northern boundary. The material had an estimated hydraulic conductivity of $6x10^{-1}$ cm/s. The wall was anchored into the bedrock and silt from 26 to 52 feet below grade.

The barrier wall was formed using a triple fluid system of jet grouting installed as overlapping circular columns. The circular columns were designed with a diameter of 3.5 feet, placed 2.5 feet on center. The finished wall met design criteria with a permeability of less than $1x10^{-5}$ cm/s.

Owner

Washington Water Power

Specialty Contractor

Hayward Baker, Inc.

Project:

Southwest Levee Cut-Off Wall 14-Mile Slough, Stockton, California]

Description:

The 14-mile slough is a body of water, contained by a levee, on the periphery of a residential area. Water egresses through sand lenses beneath the levee had caused nuisance flooding on residential properties and threatened the stability of the levee. Subsurface conditions consisted of sand, silty sand and sandy silt with two zones of excessive seepage. The first zone, at a depth of 25 feet, extended approximately 200 feet along the levee. The second zone was located 1,100 feet further along the levee and was approximately 550 feet long at a depth of 30 feet.

Jet grouting was used to construct a barrier with low permeability. A double fluid jet grouting system was used to construct soil-cement panels on 6-foot centers at an offset angle of 15 degrees from the levee centerline. The panels extended from the surface to a depth of 30 feet. Alternate panels were constructed to allow the Soilcrete to set up before adjacent panel installation.

Owner

Reclamation District 1608, Stockton, California

Specialty Contractor

Hayward Baker, Inc.

4.2.13 Advantages and Disadvantages of Jet Grouted Walls

Advantages:

- Successfully performed in a variety at soil types.
- Some obstructions encapsulated within the jet grouted soil mass.
- Performed above and below the groundwater table.
- Performed from any suitable access point.
- May be restricted to any desired depth range.
- Small diameter pre-drilled holes.
- Slanted holes may be used.

Disadvantages:

- Due to high pressure used, ground heave or movements can occur that could damage adjacent structures or utilities.
- Predrilling and supplemental probes likely required to effectively grout around boulder obstructions.
- Jet grouted column diameters and soil strength can be highly variable as they are strongly influenced by the silt and clay content of the in-situ soils.
- Contaminated spoil handling and removal required.
- Design procedures are not well established.
- Supplemental grouting may be required to fix inadequately sealed joints.
- Jet grouted columns have very little flexural strength to resist lateral pressures, which would result in frequent wales and bracing or multiple rows of ground anchors in wall systems.
- Cost is higher than other proposed wall system options.
- Multi-row wall systems needed to provide adequate width and uniformity of treatment.

4.2.14 Application

4.2.14.1 Area of Use

In the Reno Railroad Corridor, jet grouting can be constructed within the required 24-month construction schedule, used effectively as a groundwater barrier, and constructed within the trench right-of-way while still operating the UPRR. Based on areas of use alone, jet grouting is recommended for invert and wall systems in Zones 1 or 2.

4.2.14.2 Installation Procedures

The installation procedure for jet grouting requires the contractor to pre-drill borings and insert the grouting tubes, forming grouted columns during extraction. These columns would be placed contiguously along the alignment for wall construction. Whereas construction of the invert would start at the center of the trench and work toward the walls, fortifying the grouting at the wall/invert interface. Progression down the trench would continue once an entire series of columns was constructed perpendicular to the trench. With relatively rapid drilling of smaller diameter holes within the trench right-of-way, the installation procedures indicate major benefits. Based on installation procedures, jet grouting is a possible alternative method in Zone 1 or 2 for wall construction, and is a preferred method of temporary invert installation. The temporary installation would allow for a relatively dry and solid working surface to install a cast-in-place permanent invert.

4.2.14.3 Functional Performance

WALL SYSTEMS

Functionally, jet grouted walls can prevent groundwater infiltration. However, design of these wall systems to support large lateral loads is more difficult than slurry-diaphragm or secant/tangent options. In addition, these walls require secondary lateral support systems for wall heights above 8 feet and must have additional details applied to perform adequately in the freeze-thaw conditions present in Reno. Therefore, jet grouting would function as required, but it is not the leading candidate for in-place wall performance.

INVERT SYSTEMS

As a means of exposing the base of the proposed invert and providing a working surface for installation of a permanent invert solution, jet grouting can prevent groundwater infiltration and resist buoyant, uplift forces. Therefore, jet grouting would function as required, and is the leading candidate for temporary in-place invert performance.

4.2.14.4 Materials and Equipment Availability

Jet grouting can be more flexible than other construction options, oftentimes allowing contractors to change construction techniques and parameters multiple times during the life of a single project. These changes are required in locations of diverse geological conditions. Since the procedure is often iterative, it is beneficial to rely on the expertise of specialty contractors. Use of specialty contractors would reduce costs and the construction duration. Also, the use of specialty contractors ensures the availability of the construction equipment and materials, thereby reducing the amount of wasted time through inadequate construction techniques. The presence of several specialty contractors for jet grouting allows for the recommendation of this construction method for trench structural systems.

4.2.14.5 Conclusions

Jet grouting, constructed with specialty contractors, is economically feasible, produces a positive groundwater barrier that functions adequately if detailed for freeze/thaw conditions, and reduces the length of time the trains would operate on the shoofly. Therefore, with proper engineering, detailing, and construction expertise, jet grouting walls may perform adequately as a wall system for either Zone 1 or Zone 2. Furthermore, jet grouting is a leading candidate for temporary invert construction.

4.3 Permeation Grouting (Zone 1 or 2)

4.3.1 Methodology

Permeation grouting is the injection of particulate, colloidal or solution grouts into predominately granular soils. This methodology is considered for the wall, and a separate invert system of the Reno Railroad Corridor project. Grouts may include resins, silicates/emulsions, bentonite/cement and cement, although most work is done with cement based grout. The selection of the chemical type is largely dependent on the grain size distribution and permeability of the in-situ soils. Permeation grouting is a lower pressure method than jet grouting, but may still utilize pressures of several hundred pounds-per-square-inch (psi).

The grout is forced into the soil through pipes that have been strategically placed to regularly infiltrate the zone of soil to be treated. A specialized technique, *tube à machette* (a sleeved tube principle) offers more control of the grouting process. Holes, only 3/8 inch in diameter, are drilled in the grout pipe to serve as outlets for the grout. The holes and protective rubber sleeves act as one-way valves. In order to inject grout through the outlets, a double packer is inserted into the sleeve pipe. The packer is inflated to form closed seals, above and below the injection port, to allow grout to be injected repeatedly, if desired, at designated depths. The diameter of the grout bulb varies with different soil conditions. It is anticipated that the diameter will be approximately 4 ½- to 5-feet in the geology of Reno.

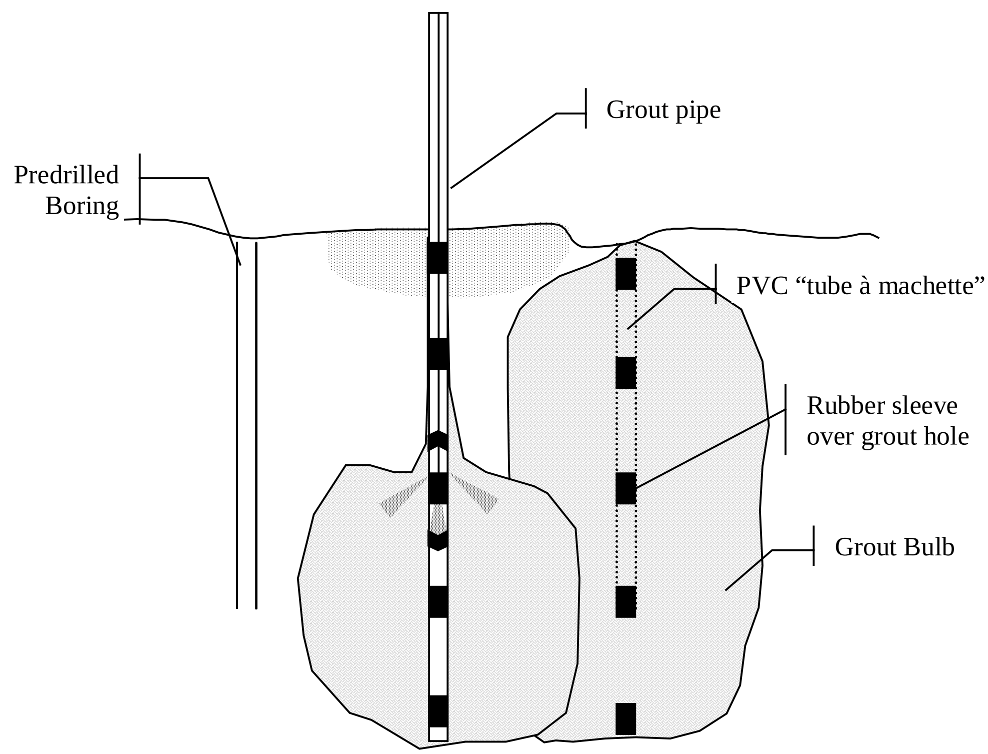


Figure 4-13 Permeation Grouting

4.3.2 Applicability to Soil Conditions

Permeation grouting is only suited for soils containing fewer than 15% fines, typically sands, gravels, and coarser open materials. Restrictions to the upper limit of fines are governed by the actual penetration of the grout in the soil stratum. With more fines, penetration is reduced; requiring bore holes placed at closer spacing relative to each other, quickly becoming cost prohibitive. The preliminary geotechnical report indicates that the City of Reno's geology contains areas of open-work gravel, sands, cobbles, and silts. With the presence of silts and some clays, some areas of the Reno geology contain more than the 15% fines recommendation; thus grout penetration would not be uniform throughout the trench alignment. Therefore, permeation grouting may be helpful in localized zones of granular materials, including gravels, sands, cobbles, and boulders, to plug leaks in other systems, but is probably not appropriate on an overall basis, for either a wall or invert system.

4.3.3 Stability of Wall Construction

A major concern of applying this technique to the construction of walls in the Reno Railroad Corridor is the stability of this system when employed adjacent to live heavy rail. However, with permeation grouting, this concern is minimized by the technique itself.

Permeation grouting is a method that is forces grout into the soil through pipes that have been strategically placed to regularly infiltrate the zone of soil to be treated. Unlike other open excavation methods, void space is dramatically reduced, actually improving bearing capacities of the treated soil. Therefore, permeation grouting is applicable in areas adjacent to live heavy rail.

4.3.4 Groundwater Control

Permeation grouting commonly creates an effective barrier against groundwater infiltration without requiring dewatering during construction. However, permeation grouting in the soils that exist in the City of Reno vicinity would most probably leave a soil mass with a larger coefficient of permeability than that of jet grouting. Soil masses with larger coefficients of permeability allow for more seepage given the same water pressures.

WALL SYSTEMS

Calculating theoretical seepage, one may determine the watertightness of a permeation grouted soil mass. The permeation process employs a top-down construction technique that reduces assurance of quality in the soil mass, therefore reducing the theoretical coefficient of permeability. Based on research[8], permeability rates for permeation grouting are on the order of $1x10^{-6}$ cm/sec to $5x10^{-6}$cm/sec. With this permeability and a mean wall thickness of 6.5 feet, these walls would seep approximately 0.13 gal/ft^2/day. Comparing the seepage rate to the evapotranspiration rate in the City of Reno vicinity ($9.0x10^{-2}$ gal/ft^2/day), these walls are virtually watertight if the thickness is increased to 9.5 feet. With

[8] James K. Mitchell, Sc.D., P.E., Professor, Virginia Polytechnic Institute and State University, Blacksburg, VA

the exception of cold, humid days, permeation grouted soil masses that are 9.5 feet thick may feel slightly moist to the touch.

INVERT SYSTEMS

Examination of a permeation grouted invert system in the Conceptual Calculations concludes that the grouted mass must be approximately twice the depth of the hydraulic head over its base. Therefore, the thickness of the permeation grouted layer will vary from a minimum of 3 feet and a maximum of 11-feet 7-inches. When comparing this thickness to the permeability analysis, it is seen that a properly constructed permeation grouted invert, in these conditions, can be made almost watertight.

However, permeation grouted invert systems are installed by grouting adjacent grouted bulbs (as shown below) creating more possibilities of localized breeches, when compared to a cast-in-place option. For these reasons, permeation grouted inverts would only provide the necessary groundwater barrier for temporary conditions of constructing a permanent invert solution.

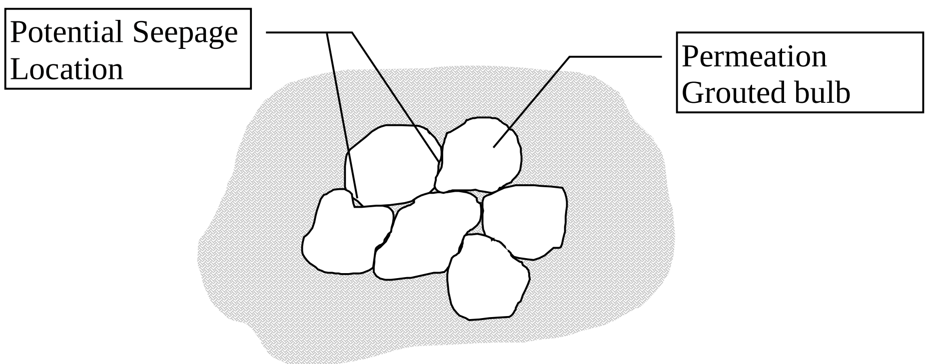


Figure 4-14 Plan View of Jet Grouted Columns

Permeation grouting to temporarily reduce groundwater infiltration during construction of a different invert system is possible. The permeation grouted soil mass would allow for manageable extraction of trapped groundwater during construction. Due to the complex geology and limited geotechnical investigation, a test section for this technique would be essential to evaluate potential seepage and actual construction rates.

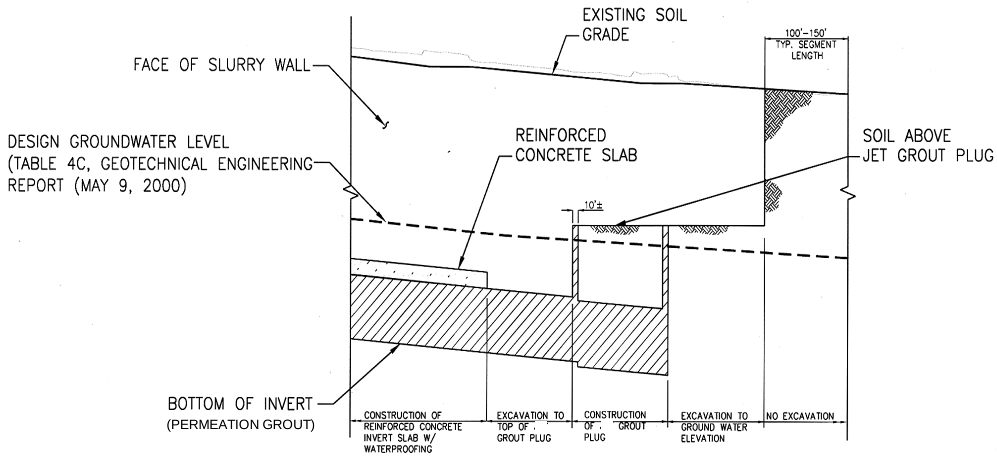


Figure 4-15 Longitudinal Section of Trench Depicting Segment Construction

Furthermore, construction of this technique for the invert system would probably be conducted in reaches of approximately 150- to 200-feet. Each reach ending with a series of contiguous permeation grouted soil bulbs extending up past the groundwater line. This segment would then be excavated and pump testing would be performed to ensure a proper seal. Remediation work may be required in the segments to correct excessive leakage. Successive segments could continue after proper sealing is achieved.

Based on an average thickness of 8-feet 6-inches, the aforementioned coefficient of permeability and a maximum head of 17 feet, the permeation grouted invert would seep approximately 0.05 gal/ft^2/day. Comparing the seepage rate to the evapotranspiration rate from above, the invert slab is virtually watertight. The performance of this method is best suited as a seepage remediation technique rather than a long-term solution.

4.3.5 Abutment Related Issues (Walls Only)

Soils stabilized by permeation grouting are brittle and have very little tension capacity when unreinforced. Therefore, when added loads imposed by bridges are greater than is practical for an unreinforced grouted soil mass, reinforcement is required. Reinforcement of permeation grouted soil masses may be accomplished through the inclusion of columns of heavy pipe sections or steel reinforcing bars in the PVC grouting pipes after grouting.

Therefore, the use of permeation grouting at the bridge locations would require additional detailing and labor. Alternatively, deep foundations may be constructed at these locations. Since deep foundations are difficult to construct in the native soils, permeation grouting is not recommended at these locations. However, a different wall system could be constructed and attached to the grouted soil masses. This connection would require grouting behind the abutment wall to seal the trench in locations below the groundwater table.

4.3.6 Duration of Construction

Much like jet grouting, the duration of construction is governed by the time required to drill the application holes. For wall construction, these holes are spaced approximately 4 feet on center. Based on the assumptions made in the jet grouting section[9], this construction would progress at a pace of 189 ft^2/shift. Therefore, based on production rates, it is anticipated that the construction of the wall sections could be completed within the target timeframe. Due to these production rates, permeation grouting can be recommended for application to either Zone 1 or Zone 2.

Similar to wall construction, the holes for the invert are spaced approximately 4 feet on center. Based on the assumptions made in the jet grouting section, this construction would progress at a pace of 37-square-feet-per-8-hour-shift (37 ft^2/shift). This production rate is unacceptable for completion of the invert within the targeted 24 months. Therefore, based on production rates, permeation grouting cannot be recommended as the primary construction method in Central Reno.

4.3.7 Traffic and Noise Impact

The traffic and noise impacts of permeation grouting are minimal, disregarding the duration of the construction. The equipment used (a drill rig, a grouting rig, a grouting pump, monitoring devices, and batching facilities) conducts the majority of the work below ground, thereby insulating the surroundings. This process has even fewer noise impacts than jet grouting due to the different equipment used to inject the grout into the soil.

The equipment is operated within the construction right-of-way, creating nominal spoils. Rather than replacing soil, the permeation process cements the existing soil together. Therefore, the spoils created by the drilling process may be stockpiled on-site and periodically hauled to disposal sites. For the purposes of this analysis, the periodic traffic increase due to hauling spoils off-site is negligible.

4.3.8 Right-Of-Way Impact (Wall Systems)

As with jet grouting, permeation grouted wall systems are certain to have right-of-way concerns. These concerns are due to the required secondary lateral support systems. In addition, like jet grouting, this wall system requires lateral bracing to span distances greater than 8 feet, thus requiring grouted ground anchors. The grouted ground anchors require the largest easement of the proposed secondary systems. However, the proposed project limits as stated in the DEIS for the Reno Railroad Corridor project provide adequate underground easements for the required grouted ground anchors. Based on the right-of-way criteria, permeation grouting is recommended for any location along the alignment.

[9] Mark Doehring, Kleinfelder, Reno, NV

4.3.9 Aesthetics (Wall Systems)

Permeation grouted walls are similar in appearance to jet grouted walls and traditional slurry-diaphragm walls, having 4- to 6-inch relief and a cemented native soil surface. As with jet grouted walls, permeation grouted walls are not as flexible as those produced with slurry-diaphragm construction. These brittle, unreinforced walls require special detailing and it is much costlier to install aesthetic treatments such as façades or facing walls. Therefore, in the areas where the aesthetic appearance of cemented native soil is unacceptable, the use of permeation grouting would be discouraged based on aesthetic appeal.

4.3.10 Conceptual Calculations

WALL SYSTEMS

Like jet grouted walls, permeation grouted walls are typically designed as gravity wall systems, however they may be designed as anchored reinforced diaphragms. For the purposes of this discussion, permeation grouted walls were conceptually designed as gravity systems.

Permeation grouted walls must be large enough to resist overturning and provide enough base friction to resist sliding. However, due to the relatively weak compressive strength of permeation grouted masses (200 psi to 2000 psi), design for flexural conditions at abutments or locations of large surcharge loads is impractical. For design purposes, it is assumed that these wall systems would produce unit weights of the permeation grouted mass on the order of 130 pcf and a coefficient of static friction of 0.35. The wall thickness in Zone 1 is typically on the order of one-half the wall height. Due to hydrostatic pressure developed behind the walls in Zone 2, the required wall thickness is greater.

In Zone 1, the average wall heights are 14.5 and 11.5 feet for the eastern end and western end, respectfully. Construction of these walls would require 7.25- and 5.75-foot-thick walls, based on an average width of 4.5 to 5 feet for permeation-grouted bulbs requiring two rows, as would be appropriate to City of Reno geology. However, to resist groundwater infiltration, the wall thickness must be increased to 9.5 feet, also requiring 2 rows of grouted bulbs.

In Zone 2, the average wall height is 28.5 feet, requiring a wall thickness of approximately 15 feet. Based on the permeation grouted column diameters expected in the City of Reno, the construction of these walls would require 3 to 4 rows of grouted bulbs.

INVERT SYSTEMS

Examination of the permeation grouted invert option required an analysis of three separate design constraints: 1) buoyancy limits, 2) practical construction criteria. Each of these constraints are described below

Determine Buoyancy Limits

The critical design for buoyancy was calculated for the condition in which the invert was constructed and excavation was completed to the top of the permeation grouted plug, thus eliminating any overburden. For this analysis, an iterative solution was used to vary the thickness of the invert plug and calculate the buoyancy and opposing forces.

As described in Groundwater Control, it is preferable to design a permeation grouted invert system as a temporary construction method, allowing for construction of a cast-in-place alternative. Therefore, different factors of safety and design criteria will be applied to this system compared to the permanent solution. Specifically, the groundwater elevation used to determine the buoyant forces was taken as the highest measured groundwater level during the yearlong monitoring conducted by Kleinfelder. Moreover, the factor of safety was reduced to be consistent with temporary construction conditions, namely 1.2. Using this criterion, buoyant forces were determined.

The buoyancy force is created through a differential head (h) between the bottom of the invert plug and the highest observed groundwater elevation. The differential head was determined using the measured distance between the top of the invert plug (determined by layout geometry) and the highest observed groundwater table elevation (determined through site exploration) and adding the trial invert thickness. The buoyancy force was determined by multiplying the differential head by the unit weight of water, subsequently increasing the value by a 20 % safety factor. Counteracting the buoyant force is the unit weight of the invert plug.

Ignoring any slab bending performance, the unit weight of the invert plug was determined by multiplying the trial invert thickness by the assumed unit weight of the permeation grouted mass (130 lb/ft^3)[10].

After determination of the opposing forces, they were compared to ensure the unit weight of the mass was greater than the design buoyant force.

These forces are extremely sensitive to fluctuations in groundwater elevations. Therefore, in final design, it is imperative that the highest probable groundwater elevation be included in these calculations. Alternatively, the elevation of the groundwater table that effects permeation grout thickness may be held constant through the employment of weep holes or other pervious detailing at the highest acceptable groundwater elevation. In a natural event that elevates the groundwater table over this threshold value, the water would flood the trench rather than fail the invert and wall system.

<u>Apply Practical Construction Criteria</u>

Application of practical construction criteria is difficult in preliminary engineering. Each construction team may approach this differently using different materials, equipment and methodology. Safety factors and site-specific geotechnical data are also a concern for developing practical construction criteria. Therefore, for the purposes of this study, it is assumed that the minimum practical thickness of permeation grouting is 3 feet.

[10] Donald A. Bruce, PhD., GEOSYSTEMS, L.P., Venetia PA

Results

Using the criteria, assumptions and procedures outlined above, minimum, maximum, and average permeation grouted plug thickness was determined and summarized below:

Minimum Thickness: 3'-0"

Maximum Thickness: 12"-9"

Average Thickness: 9'-5"

Based on the values above and the plan and profiles shown in Appendix A, the total volume of treated soil for permeation grouting is approximately 96,900 yd^3.

4.3.11 Cost

WALL SYSTEMS

Typically, permeation grouting costs $150 per treated cubic yard[11], not including secondary support systems. Based on treated materials and gravity wall designs with wall thickness of 9.5 feet and 15 feet, the costs associated with permeation grouting are $53/$ft^2$ and $83/$ft^2$ for Zones 1 and 2, respectfully. Using the per-square-foot values above, a weight average of $71/$ft^2$ is used for estimating total costs. Overcoming the shadowing effects of large boulders or the need to decrease the grout spacing for finer materials would require additional costs.

INVERT SYSTEMS

Using the same costs as those applied to the wall system, the costs associated with permeation grouting are $63/$ft^2$ - measured in plan. Overcoming the shadowing effects of large boulders or the need to decrease the grout spacing for finer materials would require additional costs.

4.3.12 History of Successful Application

Below is an example of a similar project used for a similar purpose:

Project:

Greenbelt Route, Park Road Tunnels, Washington, D.C.

Description:

The project included the construction of two stacked tunnels, 21 feet in diameter and 3,300 feet each in length. Initial dewatering proved ineffective and chemical grouting (a similar procedure to permeation grouting) was selected to reduce groundwater seepage and provide temporary ground support. Subsurface conditions ranged from coarse terrace deposits to sandy silts and silty sand. The grout consisted of sodium silicate and reagent. The system proved time effective and cost

[11] James K. Mitchell, Sc.D., P.E., Professor, Virginia Polytechnic Institute and State University, Blacksburg, VA

effective, and permitted the use of open tunnel shield in high groundwater conditions.

Owner

Washington Metropolitan Area Transit Authority (WAMATA)

Specialty Contractor

Nicholson Construction Company

4.3.13 Advantages and Disadvantages of Permeation Grouting

Advantages:

- Obstructions may be able to be encapsulated within the grouted soil mass.
- Performed above and below the groundwater table.
- Performed from any suitable access point.
- Localized use to plug leaks.
- Performs as a structural wall when reinforced.

Disadvantages:

- Generally ineffective in soils with more than 15% fines.
- Pre-drilling and supplemental probes are likely to be required to effectively grout around boulder obstructions.
- Local "windows" to groundwater may result.
- Multi-row walls needed to provide adequate width and uniformity of treatment.
- Cost

4.3.14 Application

4.3.14.1 Area of Use

Based on methodology, low resistance to lateral loads and brittle cracking in extreme temperatures, permeation grouting is only recommended in localized regions of shallow application in Zone 1 or as a secondary line of defense in Zone 2. However, permeation grouting is the recommended technique to plug leaks in the proposed walls systems, both during and after construction.

4.3.14.2 Installation Procedures

The installation procedure requires the contractor to pre-drill borings, insert the grouting tubes forming grout bulbs through a tube à machete. Major benefits of the permeation grouting technique are 1) relatively rapid drilling of smaller diameter holes and 2) keeping construction activities within the trench. Based on installation procedures, permeation grouting is recommended in either Zone 1 or Zone 2 for wall systems. On the other hand, the expanse of the invert in the Central Reno region requires an aggressive and unattainable production rate.

Based on the invert installation procedures, permeation grouting is not recommended for invert construction.

4.3.14.3 Functional Performance

WALL SYSTEMS

The functional performance of permeation grouting for wall systems in granular soils to create a groundwater barrier is excellent. However, in the application for the Reno Railroad Corridor, it is necessary for the selected system to support lateral loads, resist temperature degradation and retain soil efficiently. Based on these criteria, permeation is not recommended for locations in the project that require more than 8 feet of soil to be retained, without added surcharges or bridge loads. Therefore, permeation grouting would function adequately as a stand-alone wall system in Zone 1 only or for a method of patching seepage through a different method of either wall or invert construction

4.3.14.4 Materials and Equipment Availability

Like jet grouting, permeation grouting is not an exact science, oftentimes requiring contractors to change construction techniques multiple times during the life of a single project. These changes are required in locations of diverse geological conditions. Since the procedure is often iterative, it is beneficial to rely on the expertise of specialty contractors. Use of specialty contractors would increase construction quality. Also, the use of these specialty contractors ensures the availability of the construction equipment and materials, thereby reducing the amount of time wasted on inadequate construction techniques. The presence of specialty contractors for permeation grouting allows for the recommendation of this construction method as the primary means of patching localized groundwater *breeches in any invert system.*

4.3.14.5 Conclusions

WALL SYSTEMS

Other than shorter walls without surcharge loads, permeation grouting is impractical for use for trench walls in the Reno Railroad Corridor project. However, its functional performance in applications to stop groundwater infiltration through localized discontinuity in other wall systems is excellent. Therefore, if desired for the project, it is recommended that this be used as stand-alone walls in Zone 1 only and throughout the project as a means to provide a solution for localized wall leaks.

INVERT SYSTEMS

In Central Reno, permeation grouting is impractical for use for a trench invert due to construction time considerations. However, its functional performance in applications to stop groundwater infiltration through localized discontinuity in other invert systems is excellent. Therefore, if desired for the project, it is recommended that this be used throughout the project as a means to provide a solution for localized invert leaks.

4.4 Secant/Tangent Piles (Zone 1 or 2)

4.4.1 Methodology

Secant and tangent pile walls are a methodology considered for the wall systems of the Reno Railroad Corridor project. They consist of a continuous line of drilled piers extending below the excavation subgrade. Overlapping or cutting into adjacent drilled shafts defines secant pile wall installation. Tangent pile walls consist of shafts butting up to adjacent shafts, or shafts separated by some small distance, with the gap filled by a smaller diameter drilled pier located behind the primary row of shafts. Shafts are excavated by either wet or dry drilling techniques. Slurry or casings may be required to maintain the vertical face of the drilled shafts.

Upon completion of the shaft, steel reinforcing cages or steel beam sections are placed in the excavation, which is then filled with structural concrete. These walls can be relatively impermeable if installed properly. The initial installation of watertight tangent pile walls is difficult due to protrusions along completed adjacent shafts. However, by grouting behind the joints, these walls can be constructed to be watertight.

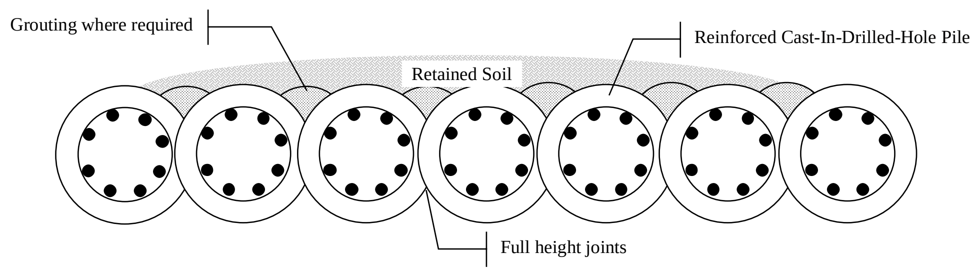


Figure 4-16 Part Plan (Secant Pile)

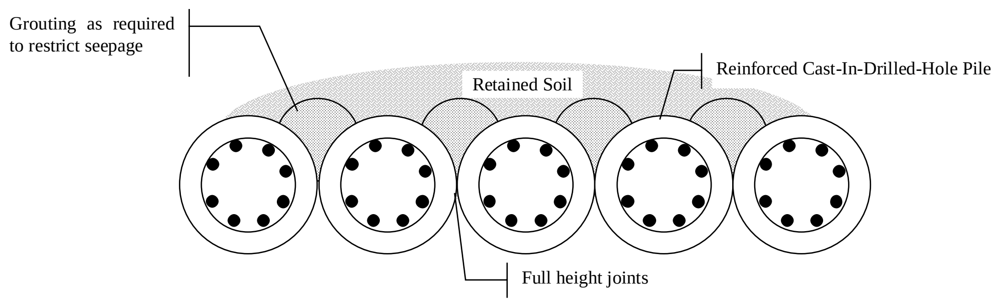


Figure 4-17 Part Plan (Tangent Pile)

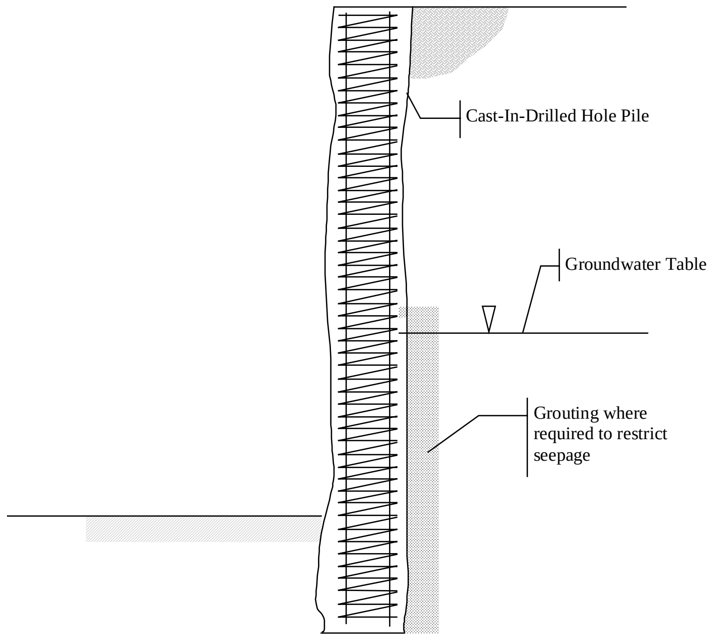


Figure 4-18 Pile Wall Section

Similar to slurry-diaphragm walls, secant and tangent piles are relatively rigid, continuous walls that can be used when it is necessary to reduce ground movements and groundwater lowering outside of the excavation. Secant and tangent pile walls can also be used for permanent support of vertical loads and facilitate top-down construction. A main difference between slurry-diaphragm walls and secant/tangent walls is that the secant/tangent walls are constructed with conventional drilled shaft equipment. These walls are more suitable than slurry-diaphragm walls for confined work areas, irregular wall geometry, or for projects requiring drilled shaft underpinning. In this regard, secant or tangent pile walls can be considered for excavation support beneath existing structures.

4.4.2 Applicability to Soil Conditions

Secant and tangent pile walls can be installed through any subsurface condition including very dense granular soils as the shafts are excavated with conventional drilled pier installation equipment. However, drilling through cobbles and boulders, such as those found in City of Reno subsurface conditions, with conventional auger-type equipment, would cause difficulties that would result in very slow construction progress. Different drilling techniques, including hydraulic casing oscillators combined with 5-ton steel rods (GADS), may progress drilling faster than traditional techniques used in the preliminary geotechnical investigation. Boulders may also cause some interference in creating successful links between adjacent piles. The resulting gaps or windows would be of particular concern in areas of the alignment below the groundwater table. However, where these inconsistencies exist, permeation grouting behind the column joints may be used to control groundwater infiltration.

4.4.3 Stability of Wall Construction

Special geotechnical consideration is required for secant-tangent walls. In the proposed methodology, excavations required for pile construction are held open using bentonite slurry until they can be backfilled with structural concrete. The outward pressure provided by the slurry, on the sides of the excavation, resists the lateral forces that cause caving. However, a major concern of applying this technique to the excavation in the Reno Railroad Corridor is the stability of this system when employed adjacent to live heavy rail.

Based on the analysis that was performed on the slurry-diaphragm wall technique, having a relatively larger open excavation, this method shows little risk of collapse or unacceptable vertical deflections.

4.4.4 Groundwater Control

Using the secant or tangent wall techniques presents minor difficulties in ensuring watertight construction. These difficulties stem from the construction method’s natural cold joints between the piles and grouted columns. Terminating wall breeches during construction is accomplished by grouting behind the wall, down through the retained soil. Similar, localized patching would be required, extending the construction schedule. This option, although more difficult to construct, offers an excellent groundwater barrier.

The coefficient of permeability for a completed and patched secant or tangent wall is approximately $1x10^{-6}$ cm/sec. With a mean wall thickness of 3.5 feet combined with 3 feet of grouting (combined thickness of 6.5 feet), the theoretical seepage rate from this wall system is less than $8.3x10^{-2}$ gal/ft^2/day. This seepage rate is counteracted by evapotranspiration, therefore becoming insignificant. Based on these values, secant or tangent pile wall construction provides watertight results. For its groundwater controlling capabilities, a secant or tangent pile wall is feasible for Zone 1 or Zone 2.

4.4.5 Abutment Related Issues

Secant or Tangent piles are constructed with reinforced drilled shafts capable of being designed to resist both vertical and lateral loads. Similar in performance characteristics to slurry-diaphragm walls, these wall systems are easily adapted to serve as both retaining structures and bridge supports. The reinforcement must be modified near bridge locations and may be aided with a secondary lateral support system.

Efficiently constructed at bridge abutment locations, secant or tangent pile walls are compatible with both struts and grouted ground anchors. Using secondary systems to support lateral loads allows for optimal wall thickness designs. Alternatively, larger pile diameters may be used where right-of-way and vertical clearance restrictions prevent the use of a secondary lateral support system.

Since the reinforcing cages are prefabricated, the construction schedule is not affected. Construction continues similarly along the entire wall alignment with the exception of the cage placement. A reinforcing cage with a stiffer bar configuration is inserted in the drilled holes at the abutment.

Additionally, secant or tangent pile walls may be selected for construction only at the abutments, supporting the structures and tying into a different wall design adjacent the bridge abutments. In this scenario, due to the design flexibility of the cast-in-drilled-hole pile structures, the connections are straightforward.

Regarding abutment issues, secant or tangent piles are appropriate for construction in Zone 1 or Zone 2.

4.4.6 Duration of Construction

The duration of construction for secant or tangent piles must be estimated by examining preliminary geotechnical results. The geotechnical investigation for the preliminary engineering encountered difficult drilling. For holes with a similar diameter to that of the proposed secant or tangent piles, drilling 55 feet in depth required between one and three days. Based on the average of 55 feet in two days, and assuming the piles are spaced 4 feet apart, on center, it is estimated that production rates in Zone 1 would be approximately 13 linear feet of wall per day, and in Zone 2 would be approximately 7 linear feet of wall per day, based on single sifts working on a single heading (190 ft^2/shift[12]). These production rates do not include the grouted columns that must be created to limit wall seepage. It is estimated that 175 linear feet of column can be produced per day. Since their production rate is significantly faster, the grouted columns only affect production by a few percent. In addition, the placement of the concrete and reinforcement will be conducted simultaneous to the drilling operation in adjacent shafts, slightly impacting the construction of only the first few. Therefore, the adjusted production rates for Zone 1 and Zone 2 are approximately 12 linear feet of wall per day and 6.5 linear feet of wall per day, respectively. Based on these production rates secant/tangent piles are recommended for Zone 1 and Zone 2.

4.4.7 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Corridor.

Two major factors affect noise and traffic impacts from construction of wall systems in the Reno Railroad Corridor: 1) equipment use and 2) duration of construction.

The following is a brief list of the major construction equipment used to construct secant or tangent pile walls:

- Drill Rig
- Concrete pump
- Grout pump
- Grout and Concrete delivery trucks (Optional)
- Materials delivery trucks
- Crane
- Spoils Hauling Truck

[12]William S. Fischetti, P.E, Malcolm Drilling Company, Vista, CA

4.4.8 Noise Impacts

As seen in the above list, these vehicles are common for construction sites. These vehicles, although not quiet, introduce only minor levels above average daily traffic in the downtown City of Reno vicinity. Additionally, these vehicles create less noise than is produced by train whistles in the current rail configuration.

An additional consideration of project duration is required at this stage of analysis. The length of construction, based on the region of application, may extend from only a few days to years. This specific construction equipment would therefore be left on-site for as many as 4 construction seasons, providing breaks for inclement weather. Overall, the equipment used to construct secant or tangent pile walls and the time frame during which the equipment would be used are expected to result in only minimal noise impacts.

4.4.9 Traffic Impacts

Secant/Tangent pile construction requires drill rigs and cranes to operate in close proximity to one another, which, combined with staging requirements, affects construction in a manner similar to that of slurry-diaphragm construction. The main difference between the traffic impacts imposed by the secant/tangent pile option and slurry-diaphragm construction is the slurry handling process. The amount of slurry used for stabilization, at one time, is greatly reduced by the secant/tangent pile method. The reduction in slurry usage is possible due to the reduced need of slurry in a single drilled shaft, compared to 8- to 24-feet wide panel used in slurry-diaphragm wall construction. Therefore, recycling of slurry may not be required, which reduces the required staging area to only 1,000 square feet. In addition, unlike slurry-diaphragm construction, the staging area may be established in a location greater than 1,000 feet away from the current construction activity.

Based on traffic impacts, the secant/tangent pile option offers a solution to minimize shoofly operation, reduce staging requirements, and reduce transportation of excavation and slurry operation spoils.

4.4.10 Right-Of-Way Impact

Secant or tangent walls would require secondary lateral support systems in locations of large surcharges, bridge loads, and high groundwater. These added forces are resisted by these secondary systems to minimize the diameter of the drilled shafts. Based on preliminary calculations, the maximum length required for underground easements for grouted ground anchors is 50 feet. Based on the *Draft Reno Railroad Corridor Environmental Impact Report*, the regions in which grouted ground anchors are anticipated provide ample underground easements. However, due to potential impacts on future development, use of grouted ground anchors is discouraged in locations other than bridge abutments (Figure 4-19).

At bridge locations, it is reasonable to use tieback anchors, since the right-of-way is directly under city streets. Other locations may be best suited for struts that brace the wall systems against each other, passing laterally across the top of the trench.

If used in Zone 1, it may not be necessary to provide the grouting behind the wall, as discussed in Groundwater Control, providing a 3-feet 6-inch structural wall thickness. This thickness is much smaller than a cantilever wall with a footing heel continually encroaching adjacent property. Therefore, in regions that do not require additional lateral support from tiebacks, the required maintenance easement is only the width of the wall.

However, in Zone 2, these walls must be grouted behind the piles from the bottom of the pile up to at least the design groundwater elevation, thereby increasing the structural wall thickness to 6-feet 6-inches below groundwater. This length, although shorter than cantilever walls, makes these walls less attractive to the project than thinner options such as slurry-diaphragms.

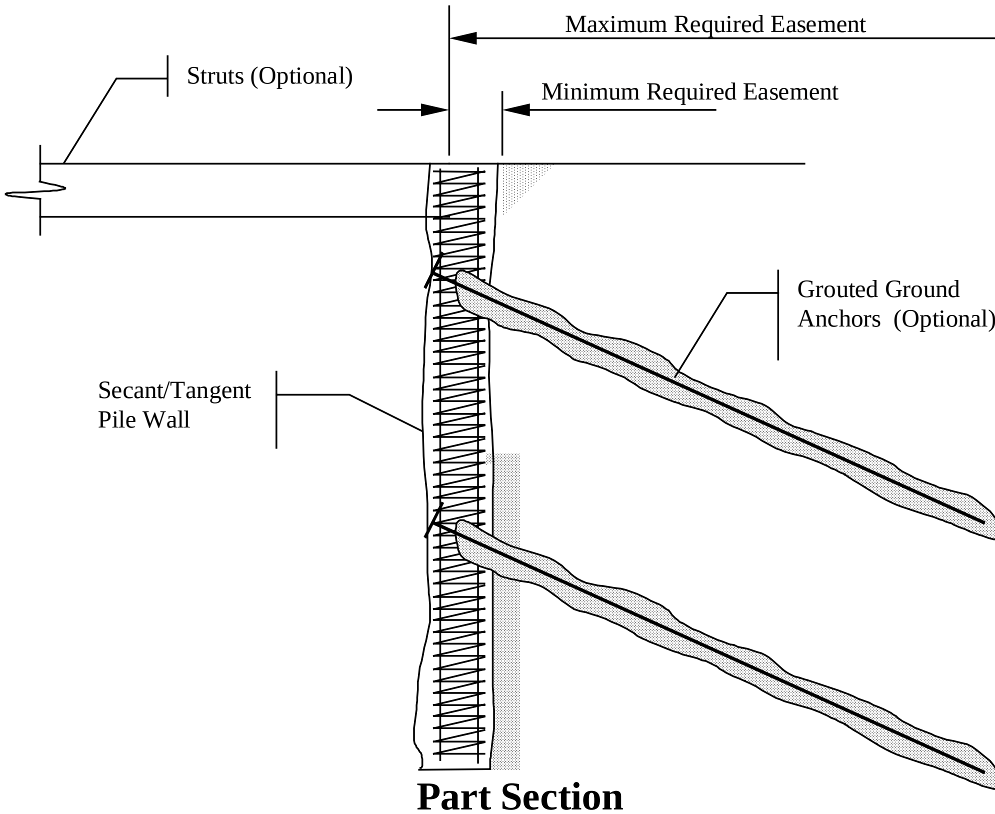


Figure 4-19 Easement Requirements

4.4.11 Aesthetics

Secant or tangent pile walls would produce a finished surface with a relief of approximately have the diameter of the pile – much more than jet and permeation grouting. However, the final wall surface is more flexible than those produced by the grouted methods. The walls produced with this construction technique are adaptable to aesthetic treatments similar to those of slurry-diaphragm walls, including façades and facing elements. The cylindrical shape of these piles is more difficult to detail for aesthetic treatments, and therefore would require more time and expense for application. Based on the techniques used at similar projects, it is estimated that aesthetic treatments would add approximately 15% to the construction schedule, costing an additional $4 to $20 per square foot of wall surface.

Although secant-tangent pile walls are easily modified for aesthetics, it is not recommended to apply either façades or facing walls. These applications would not allow for simple wall maintenance and may pose other risks associated with freeze-thaw conditions. Additionally, the facing may hide wall leak sources. These sources, if present, can be fixed by grouting behind the wall. Not visible, leaks are difficult, or impossible, to repair. Therefore, if secant-tangent pile walls are chosen it is recommended that the natural face of the wall be exposed in the final configuration.

4.4.12 Conceptual Calculations

Secant/Tangent walls are designed for the same internal forces applied to slurry-diaphragm elements. These piles are assumed to be uniaxial column elements constructed with concrete with a compressive strength of 3500 psi and using traditional steel reinforcing. Based on the maximum height, measured from the top of the trench to the top of the invert plug and assuming a depth to fixity of three pile diameters, the total shaft length is 45 feet. Using these design parameters and the internal moment of 1,169 kip•ft, these shafts would be 3.5 feet in diameter and consist of 21 (#9) reinforcing bars distributed evenly within a spiral cage. Relying on a strut at the top of the wall, the above section is capable of withstanding the forces imposed by the adjacent soil and hydrostatic pressure. The pile detail described above is simply constructed with standard techniques and fabricated with typical details. The common nature of this construction, combined with its design feasibility, allows for a recommendation in any zone.

4.4.13 Cost[13]

Based on 3.5-foot-diameter reinforced concrete shafts, combined with grouting intermediate columns, it is estimated that secant/tangent wall construction (including slurry) would cost approximately $85 per square foot of wall surface. This cost, based on typical pile construction techniques, requires 25% in additional contingencies due to difficult geology and long production rates. Therefore, to estimate total construction costs the price must be adjusted to $107/ft^2. These costs may be reduced in final construction due to the use of unique drilling techniques applied by specialty contractors.

4.4.14 History of Successful Application

The following project demonstrates the possibility of using non-traditional drilling techniques for the difficult geology at the Reno Railroad Corridor.

Project:

Allen Street Bridge – Phase 1

Description:

[13] William S. Fischetti, P.E., Malcolm Drilling Company, Vista, CA

The General Contractor constructed a pile-supported trestle work bridge across the river, complete with finger piers. This work bridge allowed for all work to be performed on the new project without interrupting the flow of traffic on the adjacent heavily traveled bridge. Due to the very fine sand deposits, the project specifications stipulated that full-depth temporary casing be used for shaft installation. The plans called for the installation of 17 (4-foot) diameter shafts and 18 (8-foot) diameter cased drilled shafts. Nine of the 8-foot diameter shafts were in the Cowlitz River. An 11-foot diameter temporary casing was installed in the river as a temporary cofferdam. After the material within this large casing was removed, the 8-foot diameter shafts were completed and seal concrete was placed at the bottom of the cofferdam casing, in order to allow the pier column work to proceed in the dry.

MDCI utilized a German-made Leffer Model VRM 2500 hydraulic casing oscillator coupled to a 120 ton Liebherr crawler crane. The oscillator exerts 5,212,000 foot-pounds of torque and is capable of oscillating the casing back and forth up to 25 degrees. Meanwhile, the Liebherr crane is still capable of full 360 degree rotation, allowing it to remove material from within the casing with a clam bucket at the same time the oscillator is advancing the casing.

MDCI has completed many successful projects utilizing the oscillator method—a method that has proven itself as a positive means of completing drilled shaft construction in difficult ground conditions while maintaining control over the stability of the hole. MDCI has capabilities to construct shafts with this method from 1 meter to 3 meters in diameter.

General Contractor:

General Construction Company

Specialty Contractor:

Malcolm Drilling Company, Inc.

Owner:

City of Kelso, WA

4.4.15 Advantages and Disadvantages of Secant and Tangent Pile Walls

Advantages:

- Potentially relatively impermeable.
- Constructed in all types of soil.
- Rigid and typically undergo significantly less lateral ground movements than flexible wall systems.
- Adaptable to confined work areas and irregular wall geometry or alignments.
- Used for permanent support of lateral and vertical loads.

Disadvantages:

- Difficult construction due to large boulder and cobbles.
- Higher cost in comparison to other wall systems.
- Difficult to construct impermeable walls with presence of small gaps between adjacent piles.
- Rough, irregular wall faces create difficult conditions for bracing members.
- Permanent architectural facing required.

4.4.16 Application

4.4.16.1 Area of Use

Secant or tangent piles are applicable in any soil type, provide an effective barrier to groundwater infiltration, and may be modified to support larger vertical and lateral loads. These walls are therefore applicable to either Zone 1 or Zone 2 in the Reno Railroad Corridor.

4.4.16.2 Installation Procedures

The major steps in the installation procedure are outlined below, highlighting the difficulties of application to the City of Reno project location.

1. Shaft Drilling-
   The shafts are drilled with a typical shaft-drilling auger. In the City of Reno soils, slurry would be used to stabilize the shaft walls during this operation. In the case of secant piles, these shafts are drilled through pre-constructed grouted columns. This is the most uncertain procedure, and difficult drilling is anticipated.
2. Slurry Placement-
   In conjunction with the shaft drilling, bentonite slurry is added to stabilize the vertical shafts from sloughing. This dense liquid is pumped from the supply staging area as far as 2,000 feet away.
3. Reinforcement Placement-
   Pre-tied reinforcement columns or wide-flange steel sections are inserted in the bentonite slurry and positioned within the middle of the shaft.
4. Tremie Concrete Placement-
   After accurate placement of the reinforcement or steel section is accomplished; structural tremie concrete is placed into the shaft. The concrete is heavier that the bentonite slurry mix. Replaced by tremie concrete, the bentonite slurry is pumped from the top of the shaft and recycled
5. Grouting Joints-
   In the case of Tangent Piles, grouting, at locations of unacceptable seepage, is completed after the concrete is placed. This grouting stops groundwater leakage through the joints.

Based on the described installation technique, the primary disadvantage of the secant/tangent walls is that difficult drilling is anticipated. Based on geotechnical exploration, this technique may be time prohibitive in large areas. However, in localized areas such as abutments, this technique may be appropriate. In such cases, the use of equipment designed for more efficient production of drilled shafts in difficult conditions may be used to improve the production schedule. Based on localized used, secant/tangent walls are appropriate in any zone.

Additional investigation into various drilling techniques may reveal solutions that make this wall type applicable throughout the trench alignment.

4.4.16.3 Functional Performance

Secant and tangent pile walls are used for hydraulic cut-off walls, soil retaining systems, and permanent structural members. Secant/Tangent walls can be internally braced with struts or tied back with ground anchors. These wall systems are effectively and efficiently used for similar applications worldwide and are anticipated to function extremely well in Zones 1 or Zone 2 of the Reno Railroad Corridor.

4.4.16.4 Materials and Equipment Availability

Specialty contractors using specialized equipment are anticipated to increase drilled shaft productivity. Reducing the construction timeframe for shaft creation is the most critical element in the recommendation of secant/tangent pile. With the correct combination of contractor and equipment, secant/tangent piles are a viable option in Zone 1or Zone 2.

4.4.16.5 Conclusions

After examination of the applicability of secant and tangent walls in the Reno Railroad Corridor project, it is concluded that these walls are appropriate to use, are possible to build, pose only scheduling difficulties, and would function as required in applications of Zone 1 or Zone 2. Additionally, secant/tangent piles are more appropriate in locations supporting large surcharge loads with limited access where slurry-diaphragm construction may not be possible.

4.5 Cast-In-Place Concrete Slab

4.5.1 Methodology

Cast-in-place concrete slabs are a methodology considered only as a permanent invert system for the Reno Railroad Corridor project. In dry locations, cast-in-place concrete slabs are constructed on a prepared surface with traditional construction methods. In below groundwater conditions, the subsurface may be sealed with tremied concrete or other temporary invert methods to provide a solid working surface capable of supporting construction equipment, materials and workers. Upon completion of the working surface, groundwater trapped within the contained area is removed and steel reinforcement mats are constructed. Forms are placed for pours of manageable quantity to maintain workability and concrete bond quality.

4.5.2 Applicability to Soil Condition

Since cast-in-place slabs are constructed on prepared surfaces, when accompanied by a seal course (tremied concrete or grouted soil mass), they are applicable to almost all soil conditions, including those present in the Reno vicinity.

4.5.3 Groundwater Control

Cast-in-place methods offer advantages not available in top-down construction. The two most prominent advantages are the ability to better control construction quality and allow for the placement of a geotextile membrane under the permanent invert. These advantages ensure that the cast-in-place slab method will provide the best water barrier of all of the examined invert methods. However, these characteristics are impossible without the installation of a temporary solution to control groundwater infiltration during construction and provide an adequate working surface below the groundwater table.

Recommended provisions for efficient removal of trapped groundwater and minor infiltration through the temporary invert include the placement of a sand-over-gravel drainage layer on the top of the temporary invert. In addition, this drainage layer is essential to provide a means of leveling the upper surface of the temporary invert and provide smoother bedding for the geotextile.

Installation of a cast-in-place invert eliminates the concerns of other inverts examined. However, attention to detail in the connection interface between the cast-in-place slab and wall systems are imperative to ensure a successful invert system.

4.5.4 Duration of Construction

Construction duration is directly related to the excavation and working surface preparation efforts. It is essential that the temporary invert is installed and the excavation work is completed to the bottom of the proposed permanent invert before placement of a cast-in-place option is possible. In addition, removal of trapped groundwater from within the contained area is required before installation of steel reinforcement mats can be completed. The construction effort can be examined in five distinct steps: 1) excavation [in excess of other options], 2) invert seal installation, 3) dewatering, 4) reinforcement installation, and 5) slab placement. With this procedure and typical production rates, it is anticipated that the concrete slab could be completed well within the target timeframe. However, the negative impacts to local surface street traffic may offset the benefits of this reduced construction schedule.

4.5.5 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Railroad Corridor.

Two major factors affect noise and traffic impacts from construction of invert systems in the Reno Corridor, namely equipment use and duration of construction.

Examinations of traffic and noise impacts on the project for the temporary methods are addressed in other sections of this document. The following is a brief list of the major construction equipment used to construct cast-in-place inverts only:

- Dewatering pumps
- Cranes
- Batching Plant
- Materials Delivery Trucks
- Concrete Pumps

4.5.6 Noise Impacts

As seen in the above list, this equipment is common in the majority of all excavation construction sites. This equipment, although not quiet, introduces only minor levels of noise above average daily traffic.

An additional consideration of project duration is required at this stage of analysis. The length of construction, 17 months, is relatively short. This construction should be completed within two construction seasons. Due to the abbreviated period of time these pieces of equipment would be in operation during the duration of the 24-month construction timeframe, the noise impact would be minimal.

4.5.7 Traffic Impacts

Since cast-in-place construction is bottom-up, fabrication procedures would require the UPRR to operate on the shoofly for a longer period than it would if top-down invert construction techniques were employed. Additionally, since large excavations are required for cast-in-place invert construction, the majority of the construction equipment would be required to operate outside the trench limits, congesting local roads with construction equipment. Among all the opportunities, the bottom-up construction of a cast-in-place concrete invert has the most severe traffic impacts.

4.5.8 Conceptual Calculations

Ignoring the seal installation procedure, examination of the cast-in-place option required an analysis of three separate design constraints: 1) buoyancy limits for the concrete invert, 2) tension limits for the reinforced concrete slab, and 3) practical construction criteria. Each of these constraints are described below.

4.5.8.1 Determine Buoyancy Limits

The critical design for buoyancy was calculated for the condition in which the invert is constructed and excavation completed to the top of the seal course (temporary invert), thus eliminating any overburden. As with the grouted options, an iterative solution was used to vary the thickness of the concrete invert plug, and then the buoyancy and opposing forces were calculated.

Unlike the grouting methods analyzed in this report, a cast-in-place option is viewed as a permanent invert solution, thereby changing the design criteria to include larger design loads and factors of safety. Specifically, the buoyancy force is calculated by using a differential head (h) between the bottom of the reinforced concrete invert and the design groundwater elevation. Subsequent increases to this buoyant force include a factor of safety of 25%. These values differ from the highest measured groundwater elevation and a factor of safety of 10% used in the temporary invert installation. Examining the weight of the cast-in-place concrete slab determines the opposing forces.

The unit weight of the invert plug was determined by multiplying the trial invert thickness by the assumed unit weight of the concrete invert (145 lb/ft3).

After determination of the opposing forces, comparisons were made to ensure the unit weight of the mass was greater than the design buoyant force. After determining the buoyancy limiting thickness of the invert plug, it is necessary to design the reinforced concrete slab.

These forces are extremely sensitive to fluctuations in groundwater elevations. Therefore, in final design, it is imperative that the highest probable groundwater elevation be included in these calculations. Alternatively, the elevation of the groundwater table that effects invert thickness may be held constant through the employment of weep holes or other pervious detailing at the highest acceptable groundwater elevation. In a natural event that elevates the groundwater table over this threshold value, the water would flood the trench rather than fail the invert and wall system.

4.5.8.2 Determine Tension Limits

After determination of the required concrete thickness to satisfy buoyancy requirements, attention is directed to the design of reinforcement in the concrete slab. For the purposes of this examination, it was assumed that the reinforced concrete slab would be designed to resist the internal forces applied by the groundwater table, as if the seal course had failed. The resulting internal forces are counteracted through proper design and detailing of the reinforced concrete slab (ACI 318). For design purposes, a specified concrete compressive strength of 3,500 psi was used. In addition, 4 inches was assumed for the clear cover distance between the extreme concrete fibers and the centroid of the tension reinforcement.

4.5.8.3 Apply Practical Construction Criteria

Once installed, the reinforced concrete invert must support live rail loads while maintaining a watertight seal. Therefore, cracking must be minimized in the design. For this purpose, the minimum thickness of the invert was determined to be 2 feet. This thickness allows for the support of the train loads while maintaining enough integrity to ensure watertightness.

4.5.8.4 Results

Using the criteria, assumptions and procedures outlined above, minimum, maximum, and average reinforced concrete slab thickness and volumes were determined and summarized below:

Minimum Thickness:	2’-0”
Maximum Thickness:	5’-3”
Average Thickness:	3’-3”
Total Volume:	35,600 yd^3

4.5.9 Cost

The typical cost for structural concrete is approximately $175 per cubic yard ($175/yd^3) with an additional cost of $1.50 per pound of reinforcing steel. Using an estimated concrete volume of 35,600yd^3 and 2.4 million pounds of reinforcing steel, the total average cost of the invert is $34/$ft^2$ – measured in plan.

4.5.10 History of Successful Application

Traditional cast-in-place reinforced concrete slabs over tremied concrete seals have been successfully used throughout the world for similar and more difficult conditions. The list of these successes is too long to reproduce.

4.5.11 Advantages and Disadvantages of Cast-In-Place Concrete Slabs

Advantages:

- Applicable in any soil condition
- Traditional construction techniques
- Relatively high construction productivity rates
- Established design standards
- Provides secondary line of defense against infiltration

Disadvantages:

- Bottom-up construction requires work outside the trench right-of-way
- Cost
- Removal of trapped groundwater from the contained area required before installation.

4.5.12 Application

4.5.13 Area of Use

In Central Reno, a cast-in-place reinforced concrete invert slab can be constructed within the required 24-month construction schedule and used effectively as a groundwater barrier. However, due to the bottom-up nature of this method, it may be difficult to confine the construction activities to limits within the trench right-of-way, thereby increasing traffic impacts.

4.5.13.1 Installation Procedures

The installation procedure requires the contractor to excavate the trench section to the top elevation of the seal coarse, remove trapped groundwater from within the contained area, and then fabricate the reinforcement mats for the final slab. The installation of this technique can be completed within the required timeframe and is a standard form of heavy construction, thereby reducing installation risk. Therefore, this technique can be recommended for the permanent invert construction of the trench.

4.5.13.2 Functional Performance

The functional performance of a cast-in-place reinforced concrete slab is excellent, and offers more flexibility (with respect to supporting applied loads) than other options. Based on this criterion, a cast-in-place concrete slab can be recommended for this project and is a leading candidate for permanent invert construction.

4.5.13.3 Materials and Equipment Availability

Unlike jet or permeation grouting, cast-in-place concrete construction has well defined and easily controllable construction details, eliminating the need for contractors to change construction techniques multiple times during the life of a single project. Since the procedure is not iterative, it is not necessary to rely on specialty contractors, although, use of specialty contractors would increase construction quality. However, the use of these specialty contractors is not necessary to ensure the availability of the construction equipment and materials. Since this construction technique is common and performed with common equipment, it can be recommended for this project.

4.5.13.4 Conclusions

In Central Reno, a cast-in-place reinforced concrete slab is a practical and cost effective invert solution that eliminates the concerns of other invert options examined. Therefore, it is recommended that this be used as the permanent invert system in conjunction with a temporary system examined in this report.

5 Above Groundwater Methods

The Reno Railroad Corridor requires soil retaining solutions for both wet (below groundwater) and dry (above groundwater) configurations. The possible applications of retaining systems for dry conditions are more numerous than for the wet section. All of the wall systems described above are applicable in the dry section as well as the wet section. The additional wall systems for consideration in the dry section include:

- Cantilever Walls
- Mechanically Stabilized Earth
- Micropiling
- Soil Nailing
- Soldier Piles
- Stresswalls

The following section includes a detailed analysis of these wall systems.

5.1 Cantilever Walls (Zone 1)

5.1.1 Methodology

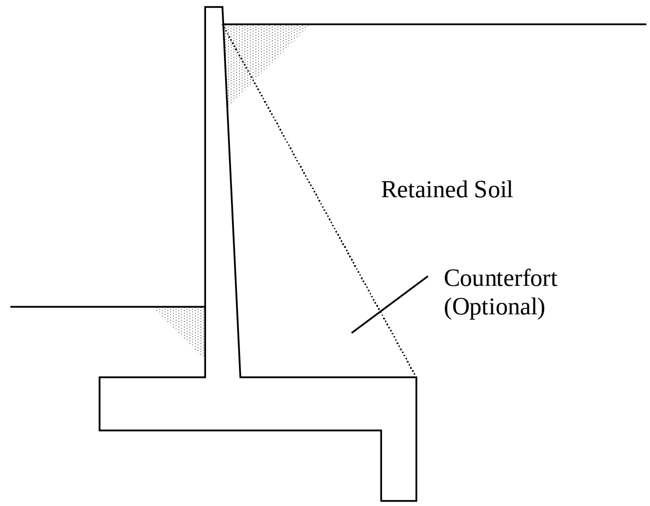
Cantilever walls are cast-in-place reinforced concrete structures. These wall systems consist of a wall stem and footing that are constructed in stages from the bottom up. Cantilever wall systems are common structures designed to resist overturning, sliding, and sinking.

Resistance to overturning is accomplished by the self weight of the soil mass resting on the heel of the footing. Therefore, the heel of the wall must get larger as the lateral forces increase. Increases in lateral force are typically due to larger walls retaining more soil, hydrostatic pressure, and surcharge loads acting on the adjacent retaining wall. To resist overturning with self-weight, the stem wall must be designed to withstand the bending moment that is applied by the lateral loads. Resistance to the bending forces is accomplished by installing steel reinforcement. The main reinforcement is set, in vertical planes, parallel to the sloping face. If necessary, counterforts or buttresses can be used to provide additional stability and reduce the stem thickness.

The stem is designed with a thicker section at the bottom, the location of highest internal moment. The thickness of this section is only needed at the bottom; thus allowing the thickness to be reduced as the wall extends above the footing. Theoretically, there is no moment at the top of the wall. Typically walls are detailed with a minimum thickness on the order of 11 inches.

The size of the base slab is selected to meet requirements for resisting overturning and sliding and to keep the pressure on the soil within the allowable bearing capacity. If the flat bottom of the slab does not provide sufficient friction against sliding, a keyway may be constructed as part of the base. Alternatively, a strut may be provided at the base to resist the residual sliding forces altogether.

Free draining backfill is usually placed behind cantilever walls to prevent water pressures from building up behind the walls. However, cantilever walls may be designed to resist hydrostatic forces and in that case no drain material would be provided.



Typical Section

5.1.2 Applicability to Soil Conditions

Cantilever construction utilizes "bottom up" construction. Bottom up construction consists of excavating soil to reach the bottom of the footing before any construction of the wall can proceed. In these cut situations, walls cannot be constructed without temporary support of the soil. Since the walls are built after the excavation, they are not as sensitive to soil type as other systems. The native soils exert common earth pressures on these walls.

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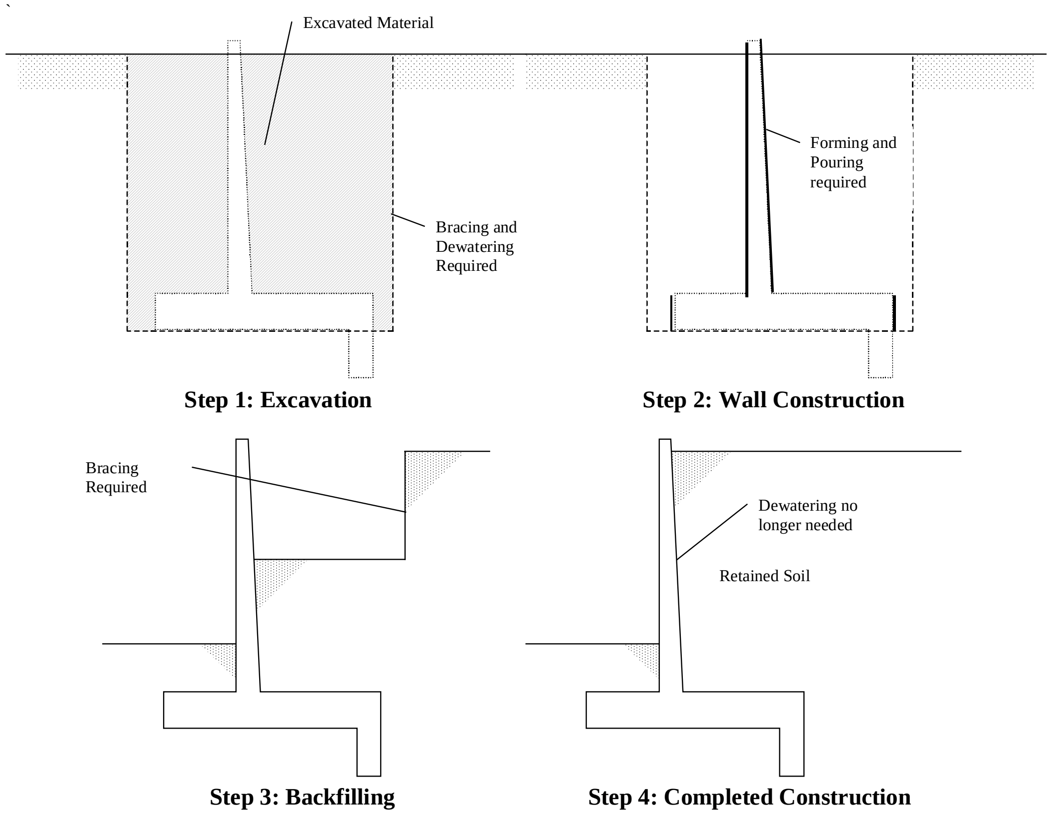


"Bottom-Up" Cantilever Wall Construction

5.1.3 Stability of Wall Construction

Due to the space requirements of bottom-up construction for cantilever walls, it is not anticipated that these walls will be constructed in regions where live rail will be running in close proximity to heel side of footing. Therefore, the stability of wall construction is not influenced by the operation of the UPRR on the shoofly during construction. This technique has minimal risk of instability.

5.1.4 Abutment Related Issues

Cantilever retaining walls are well suited to double as bridge abutment structures. Standard design practice often uses high-cantilever walls to support bridges. These wall structures are easily adapted to resist the bridge-imposed forces. In areas of a high water table, the large lateral forces applied to the structure would require a secondary support structure to help minimize the wall thickness. Details for the structural members comprising the reinforced concrete cantilever retaining wall are provided in the Conceptual Calculations.

5.1.5 Duration of Construction

Construction of traditional cantilever retaining walls is common. The ordinary nature of this technique helps reduce the construction timeframe. Use of typical details and common equipment ensure expedient development. Since dewatering is required to construct traditional cantilever walls below the groundwater table, and dewatering the City of Reno site is infeasible, a construction schedule for only the above groundwater condition is presented.

Based on a single crew working on the wall system, the approximate production rate to complete a reinforced concrete retaining wall with an average design height of 8 is 1,067 ft^2/shift. Acceleration of the total project schedule may be possible if a single crew were working on the east heading simultaneous to a single crew working on the west heading. These production rates do not include construction of secondary systems or bridge abutments. An additional 10% to 20% should be included in the schedule to incorporate these additional operations.

5.1.6 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Railroad Corridor.

Two major factors affect noise and traffic impacts from construction of wall systems in the Reno Railroad Corridor: 1) equipment use and 2) duration of construction.

The following is a brief list of the major construction equipment used to construct cantilever retaining walls:

- Backhoe
- Front loader
- Soil hauling trucks
- Concrete delivery trucks
- Concrete pumping crane
- Sheepsfoot roller
- Materials delivery trucks

5.1.6.1 Noise Impacts

As seen in the above list, this equipment is common in the majority of all excavation construction sites. This equipment, although not quiet, introduce only minor levels of noise above average daily traffic. Additionally, this equipment creates less noise than is produced by train whistles in the current rail configuration.

An additional consideration of project duration is required at this stage of analysis. The length of construction, ranging from 2.5 to 5 months, is relatively short. This construction should be completed within one construction season. Due to the abbreviated period of time these pieces of equipment would be in operation during the duration of the 24-month construction timeframe, the noise impact would be minimal.

5.1.6.2 Traffic Impacts

Since cantilever construction is bottom-up, walls over 15 feet tall would require the UPRR to operate on the shoofly for a longer period than it would if top-down wall construction techniques were employed. Additionally, since large excavations are required for cantilever wall construction, excavations would have to extend into adjacent property and possibly city streets. Among all the options, bottom-up construction has the worst traffic impacts.

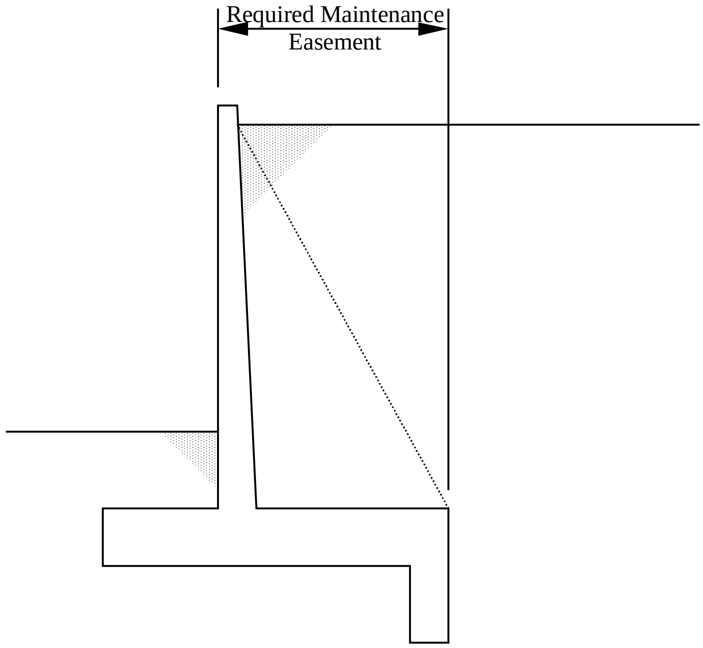
5.1.7 Right-Of-Way Impact

To fully understand the impacts that cantilever retaining walls pose, we must examine the construction configuration and the need for a secondary support system. Since the Reno Railroad Corridor runs directly adjacent to some buildings and parking lots in the above groundwater portion, the wall systems must be designed to withstand additional loading. Increasing the structural section of the wall or installing grouted ground anchors or external struts may resist this additional loading. These secondary systems, the encroachment of adjacent properties by the heel portion of a cantilever wall footing, and the required excavation clearances would affect both easements and rights-of-way. It is not anticipated that secondary systems would be required for an application of cantilever walls in Zone 1. However, if a secondary system were required, right-of-way impacts would have to be evaluated.

Without a secondary system, an easement must be maintained to provide adequate assurance that the structural system may be maintained. For the case of the retaining wall with design heights ranging from 6 to 12 feet without a surcharge, the permanent underground easement would range from about 3 feet to 6.5 feet behind the face of the wall. For a retaining wall resisting the additional surcharge from parking lots or buildings, the easement must be approximately 5 to 9 feet. This easement is smaller than that required if tieback anchors are implemented. If tieback anchors are implemented, the required easement is approximately 20 to 30 feet. With these requirements, if a secondary system were required, struts would be recommended to minimize the easement required behind the wall.

The strut system supports the front face of the wall by strutting between the two wall surfaces on the interior of the trench. The required setback using struts is approximately 3.5 to 6.5 feet.

Based on the *Draft Reno Railroad Corridor Environmental Impact Statement,* ample right-of-way is available for cantilever wall construction.



Typical Section

5.1.8 Aesthetics

In the realm of retaining structures, reinforced concrete retaining walls are among the most easily modified finished surfaces available. Multiple options are available to reduce the visual impact of concrete wall faces. Paint, epoxy, facades, and impregnation of color or texture are possible aesthetic modifications. Additionally, form liners may be employed to provide the wall with artistic designs, feature lines, or other variations of style. The estimated costs associated with these treatments are between $\$2/ft^2$ and $\$15/ft^2$. However, each added step in the construction of these wall systems has scheduling impacts that may lengthen the construction timeframe by as much as 15%.

5.1.9 Conceptual Calculations

Using the criteria defined in the Introduction, the following is a table of results based on the specified compressive stress for concrete of 3,250 psi, without a surcharge load:

Wall Configuration (Zone 1):

H	**Wall Thickness** **At the base of the wall**	**Heel**	**Toe**	**Key Required*?**

5 ft.	0.75 feet	3.0 feet	0 feet	No
10 ft.	1.0 feet	3.75 feet	1.75 feet	Yes
15 ft.	1.5 feet	5.75 feet	5.0 feet	Yes
20 ft.	2.25 feet	8 feet	9.5 feet	Yes

***Note**: If constructed adjacent to a reinforced concrete or jet grouted invert, a key is not required

5.1.10 Cost[14]

Based on construction cost data developed on many cantilever retaining walls, costs can be estimated as $35 per square foot of wall face. This per-square-foot costs does not include earthwork or special handling of contaminated soil.

5.1.11 History of Successful Application

Traditional cast-in-place reinforced concrete retaining walls have been successfully used throughout the world for similar and more difficult conditions. The list of these successes is too long to reproduce.

5.1.12 Advantages and Disadvantages of Cantilever Retaining Walls

Advantages:

- A conventional wall system with well-established design procedures.
- Good quality control can be exercised over construction methods and materials compared to other wall types.
- Adaptable to bridge abutments.

Disadvantages:

- Prohibits UPRR from operating on existing track during construction of this wall type
- Cannot be constructed below groundwater table without dewatering.
- Requires a temporary sloped or shored excavation.
- Requires a relatively long construction period to excavate, place formwork, pour concrete, cure concrete, remove formwork, and backfill behind wall.
- Requires a significant right-of-way easement for excavation and construction of walls.
- Rigid and sensitive to differential settlements.

[14] Caltrans Historic Data 1994 through 1999

5.1.13 Application

5.1.13.1 Area of Use

Construction dewatering is physically and financially prohibited throughout the Reno Railroad Corridor. These restrictions, coupled with the requirement for bottom-up dry construction, negate the use of reinforced concrete cantilever retaining walls in the regions below the groundwater table (Zone 2). However, these walls are candidates for construction above the groundwater table. Additionally, secondary lateral support systems may be used to increase the lateral capacity of the walls near bridges and surcharge loads due to parking lots and adjacent buildings. These walls are technically and financially feasible for construction in Zone 1 only.

5.1.13.2 Installation Procedures

Specific installation procedures are required based on location. Reinforced concrete retaining walls are a typical form of soil retention and the construction technique to build these systems is well defined. In the native soils throughout the Reno vicinity, these techniques must be modified to include provisions for large boulders and groundwater control.

Following is a brief outline of major events during the installation of cantilever walls:

1. UPRR Transferred to Shoofly-
   The Union Pacific Railroad must be moved to the shoofly for construction of these walls, possibly requiring UPRR to run on the shoofly longer than other options.
2. Excavation and Bracing-
   Walls are constructed from the bottom up. This requires the contractor to excavate the trench in the location of the cantilever walls. Deep excavations must be braced or laid back to avoid sloughing of the vertical soil face and, in some instances, provide contractor safety.
3. Void Filling-
   Large obstructions and poor soil conditions encountered during excavation must be corrected to ensure a level, structurally sound bottom surface. Filling the voids left behind large boulders or local over-excavation is typically accomplished with placement of a lean concrete mix.
4. Forming-
   The footing and wall forms must be placed to control the concrete during concrete placement.
5. Reinforcement Placement-
   Reinforcement is placed per plan.
6. Concrete Placement and Curing-
   Concrete is poured and cured to a specified strength or prescribed time limit. No additional work may be done on the wall system during curing.

7. <u>Form Removal-</u>
   Forms are removed and localized imperfections are repaired.

8. <u>Structural Backfilling-</u>
   Drainage material and a structural grade soil material are placed behind the wall. This process requires compaction of the new soil. Compaction is accomplished with heavy equipment far from the wall and is finished with hand compacting devices close to the wall.

9. <u>Surface Finishing-</u>
   Exposed surfaces of the walls are finished to ensure a neat appearance. Depending on the aesthetic specifications, this process may be time consuming.

Providing a contingency plan for filling voids and planning for obstructions are unique features of cantilever wall installation techniques. Such contingencies can be determined in advance through the cooperation of the City of Reno and the contractor. Therefore, minor obstacles do not prohibit this technique from being used in the Reno Railroad Corridor.

5.1.13.3 Functional Performance

Thousands of miles of cantilever retaining walls have been constructed in the United States, including the City of Reno. These wall systems are predictable and perform their required function extremely well. Cantilever wall failures are exceptionally rare and the cantilever walls proposed for the City of Reno would pose no threat to public safety. These walls, in Zone 1, are expected to perform adequately.

5.1.13.4 Materials and Equipment Availability

Reinforced concrete cantilever retaining walls require the equipment outlined in the Traffic and Noise impact section. This equipment is available for transportation to the to construction site. Additionally, the materials, namely, concrete, reinforcing steel, and wood (for forms) are common and readily available within the City of Reno vicinity. The availability of both equipment and materials makes this wall system a possible choice for the Reno Railroad Corridor Project.

5.1.13.5 Conclusions

After a thorough examination of the applicability of a reinforced concrete cantilever retaining wall system in the Reno Railroad Corridor project, it is concluded that these walls are appropriate to use, are possible to build, pose no fatal flaw, and would function as required in Zone 1 where they could be economically constructed with design heights below 15 feet.

5.2 Mechanically Stabilized Earth Walls (Zone 1)

5.2.1 Methodology

The mechanically stabilized wall system consists of multiple layers of inclusions (reinforcing elements) and select compacted backfill placed in alternately horizontal layers. Typically, the wall is completed with some type of facing system. The basic concept of the MSE wall is presented in Figure 5-1.

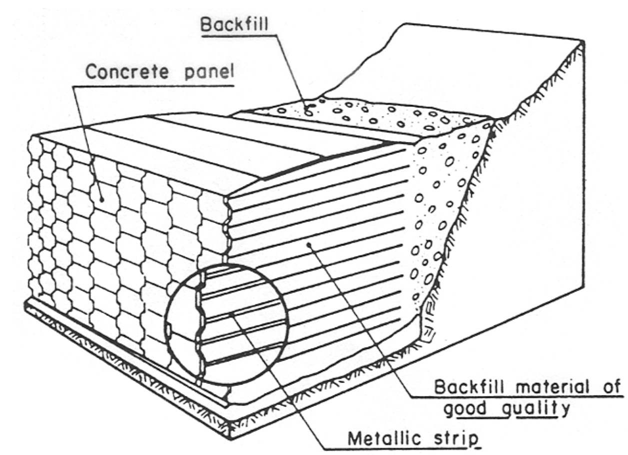


Figure 5-1 MSE Construction

The inclusions consist of continuous or semi-continuous layers of geotextile, geogrid or welded wire fabric. In terms of stress/strain behavior, inclusions may be considered inextensible (metallic) or extensible (polymeric). Inextensible inclusions deform less than the backfill material at failure. The deformation of an extensive inclusion is similar to or greater than the deformation of the backfill at failure.

Stresses are transmitted between the soil and inclusions by two mechanisms. These mechanisms are friction along the soil-reinforcement face and passive soil resistance developed along the transverse sections of the inclusion. The contribution of each transfer mechanism for a particular inclusion would depend on its roughness, geometry, and elongation characteristics. To mobilize shear stress, a relative displacement of the reinforcement with respect to the earth is required. Displacements are restrained in the direction of the strips, causing the reinforced mass to behave like a cohesive anisotropic material.

The wall facing system is optional and is constructed by either wrapping each layer of reinforcement around the overlying layer of backfill or by directly attaching the reinforcement to a pre-manufactured facing element. Wall facing elements can be precast concrete panels, dry cast modular blocks, welded wire mesh, shotcrete, or wrapped sheets of geosynthetics.

5.2.2 Applicability to Soil Conditions

Subsurface conditions along the proposed alternative are comprised of a surface layer of unengineered fill over dense glacial outwash materials. The outwash materials consist of interbedded, discontinuous layers of clay, sand, and gravel with cobbles and boulders. The soils are generally favorable to MSE wall construction above the groundwater table where there is sufficient right-of-way to construct the walls.

However, there are layers where the materials would not meet reinforced fill criteria and some processing would be required. All oversized material greater than 4 inches in diameter would need to be screened off and removed or crushed for reuse. For geosynthetics, epoxy coated and PVC coated reinforcements, the maximum particle size should be limited to ¾ inch. Fine-grained materials would need to be blended with granular soils until less than 15% by weight passes the minus 200 sieve and the plasticity index (PI) does not exceed six. Where space permits a reinforced soil slope (RSS), the fill criteria may be relaxed.

Laboratory testing of electrochemical properties for resistivity and sulfates varied from 650 to 17500 ohm-cm and 20 to 300 PPM, respectively. For steel reinforcement, FHWA recommends that backfill material exhibit a minimum resistivity of 3,000 ohm-cm and a maximum sulfate content of 200 PPM. If steel reinforcement is used some blending of site soils would be required to meet FHWA criteria.

If geosynthetic reinforcements are planned, the limits for electrochemical criteria would depend upon the polymer. The site soils meet FHWA criteria for pH requirements for polyester (PET) and polyolefin (PP and HDPE) geosynthetic reinforcements.

5.2.3 Stability of Wall Construction

Due to the space requirements of bottom-up construction for MSE walls, it is not anticipated that these walls will be constructed in regions where live rail will be running in close proximity to the required excavation. Therefore, the stability of wall construction is not influenced by the operation of the UPRR on the shoofly during construction. This technique has minimal risk of instability.

5.2.4 Abutment-Related Issues

MSE walls are commonly used as slope stabilization methods that resist the loads imposed by bridges and provide a vertical, or near vertical, face adjacent to abutments. Typical details and construction techniques are available. Therefore, features of mechanically stabilized embankments encourage the use of these systems at bridge locations.

5.2.5 Duration of Construction

Proven construction techniques have established MSE wall construction as an emergency method for global slope stabilization. Prompt construction is attributed to standard details, common equipment, common materials, and years of construction experience.

Based on a single crew working on the wall system, the approximate production rate of construction is 1,800 ft^2 per day[15]. Acceleration for the completing the proposed wall schedule may be possible if a single crew were working on the east heading simultaneous to a single crew working on the west heading.

5.2.6 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Railroad Corridor.

Two major factors affect noise and traffic impacts from construction of wall systems in the Reno Railroad Corridor: 1) equipment use and 2) duration of construction.

The following is a brief list of the major construction equipment used to construct slurry-diaphragm walls:

- Front Loader
- Sheepsfoot Compactor
- Materials delivery trucks
- Spoils Hauling Truck

5.2.6.1 Noise Impacts

The vehicles in the above list are common for construction sites. Although not quiet, they introduce only minor levels of noise above average daily traffic. Additionally, these vehicles create less noise than is produced by train whistles in the current rail configuration.

An additional consideration of project duration is required at this stage of analysis. The length of construction, based on the region of application may be only a few days to months. The desired trench construction schedule is 24 months, thereby leaving this specific construction equipment on-site for as little as one construction season. Overall, this is among the quietest of the proposed methods.

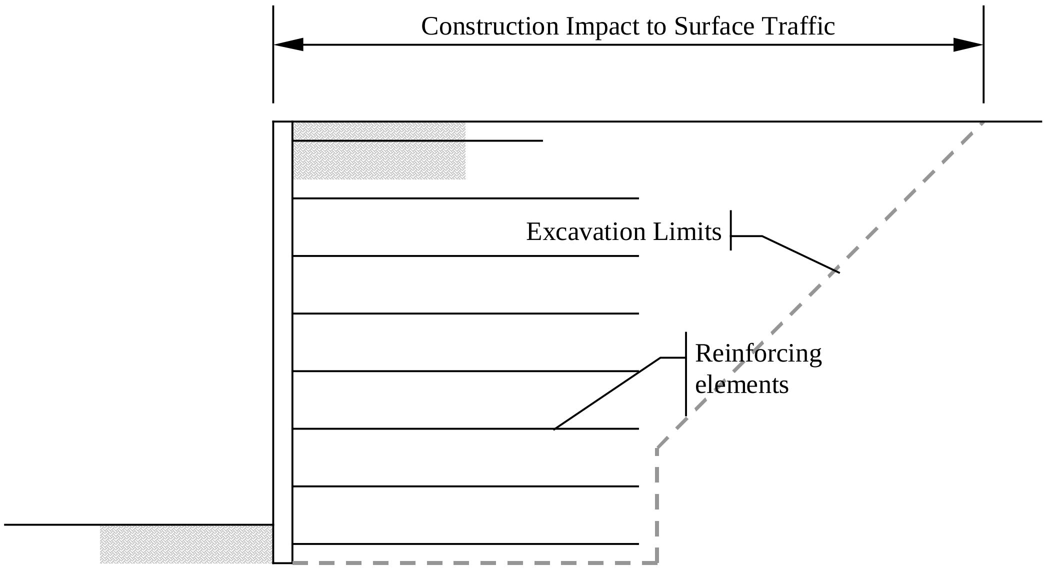
5.2.6.2 Traffic Impacts

Traffic impacts consist of congestion on local roads, closing of adjacent streets, and use of construction detours around the land occupied by a large excavation.

The most significant impact to traffic is the closing of local roads and detouring of local traffic caused by large cut sections required for MSE installation. Since MSE walls are constructed from the bottom up, the excavation must expose the base of the wall, for a distance behind the wall approximately three-quarters of the total wall height. Therefore, in regions of wall heights on the order of 20 feet, the excavation must expose approximately 15 feet in width at the bottom. Additionally, bracing of the excavation is required for vertical faces over 4 feet in height, or the contractor must lay the soil back at a one-to-one maximum slope from the 4-foot level upward. By laying back more soil, the total affected region is approximately 26 feet, measured from the inside face of the trench (as shown below). This extra construction area must be clear of any obstructions, thereby

[15] John Babcock, Transwall Earth Retaining Systems, Ogden, UT

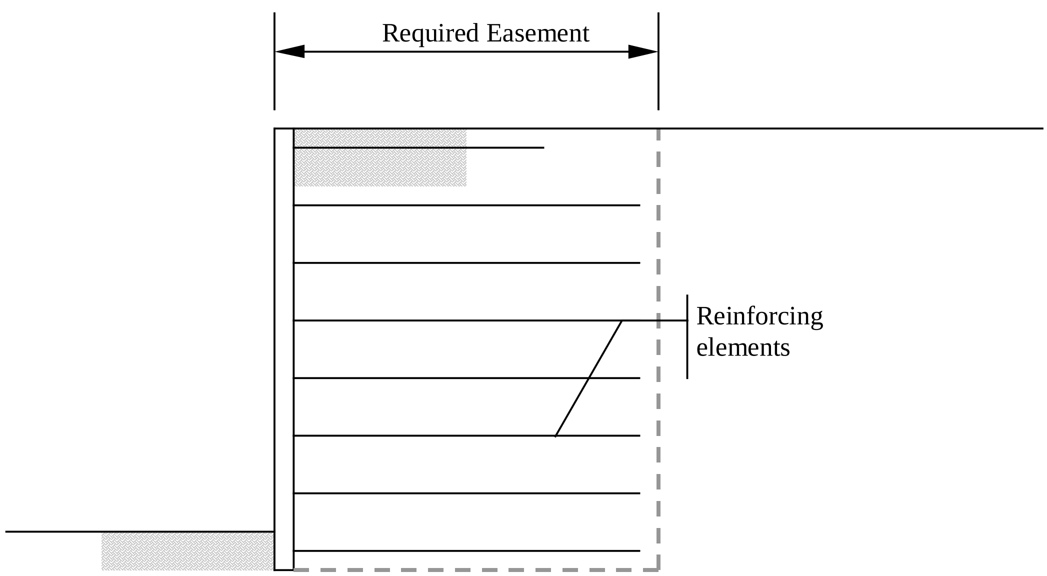
closing adjacent roads and parking lots, restricting access to local businesses, and transferring UPRR to the shoofly.



MSE Traffic Impacts

5.2.7 Right-Of-Way Impact

Mechanically stabilized earth embankments are reinforced with inclusions to increase the internal shear capacity of the soil mass. These inclusions extend into the retained soil a distance of approximately 70% of the wall height (e.g., a wall height of 8 feet would require inclusions 5.6 feet in length). As depicted in the diagram below, these inclusions are a concern for right-of-way and utility construction impacts.



MSE Right-of-Way Impacts

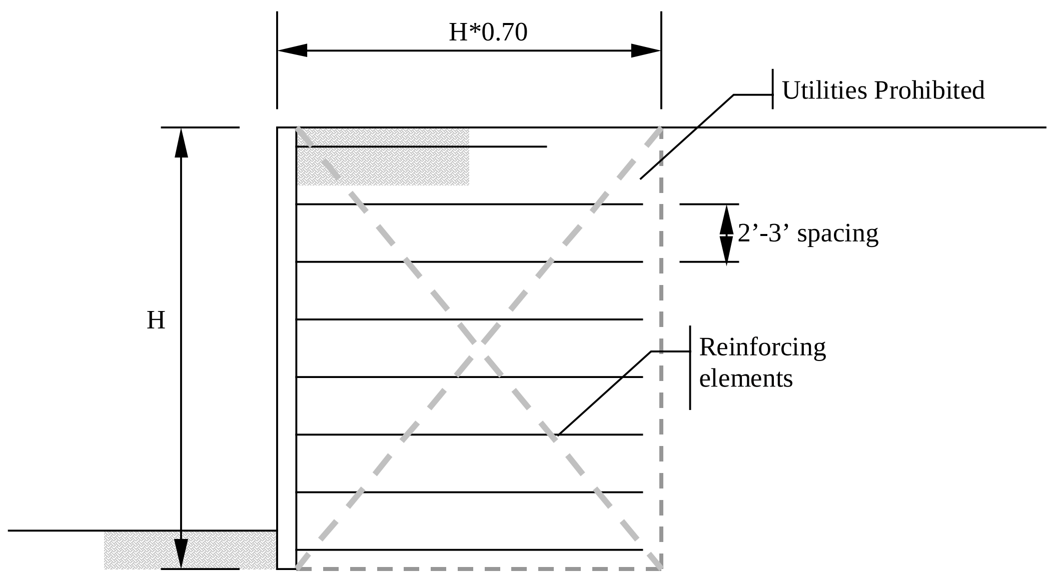
MSE wall construction is recommended where the right-of-way beyond the trench wall is significant enough to encapsulate the inclusions, thereby allowing for the possibility of maintenance, if required. Due to these requirements, these walls would be appropriate in the regions between 1) the western trench terminus and Keystone Avenue and 2) Lake Street and the eastern trench terminus.

5.2.8 Aesthetics

MSE walls require facing. Commonly, this required facing is precast concrete construction. Alternatively, the facing may be shotcreted burlap or similar treatment. Either of these applications has the flexibility of being painted, epoxied, impregnated with color, textured, or vegetated. Multiple options are available to reduce the visual impact of over 3 miles of concrete wall face. For aesthetic appeal, MSE wall construction has more aesthetic appeal than soil nailing or soldiers piles, and often provides more flexibility than cantilever retaining walls.

5.2.9 Conceptual Calculations

Based on traditional MSE wall design, a basic rule of thumb may be used to determine design and associated impacts. Inclusions must extend behind walls over 3 feet in height as these soil masses are unable to stand vertically. These inclusions are typically a minimum length of 4 feet and vary in length based on vertical location in the wall. The lengths of these inclusions are typically 70% of the design wall height, and they are placed at 2 to 3 feet vertically.



MSE Right-of-Way Impacts

Based on a maximum height of 22 feet, in Zone 1, the following table indicates the distance from the top of the wall to the reinforcing strip (z) and the length of the associated strip:

Strip No.	z	Length
1	3 feet	21.0 feet
2	6 feet	19.0 feet
3	9 feet	17.0 feet
4	12 feet	16.25 feet
5	15 feet	16.25 feet
6	18 feet	16.25 feet
7	21 feet	16.25 feet

Based on the above design, it is paramount that existing utility coordination is exhaustive.

5.2.10 Cost[16]

Based on historical cost data, MSE wall construction is approximately $42 per square foot of wall face.

5.2.11 History of Successful Application

The following section contains a brief overview of similar successful projects with this soil retention system:

Project:

I-80/Sparks Boulevard Interchange, Sparks, Nevada (Contract AC-IR-DE-0068(801))

Description:

The project involved the construction of approximately 1,000 linear feet (84,200 ft^2) of MSE wall for the bridge structure at the I-80/ Sparks Boulevard Interchange. The wall has a maximum height of 32 feet and width of 59 feet 9 inches at the bridge abutments and 31 feet 6 inches at the approaches. The wall has approximately 65,500 yd^3 of granular fill. The subgrade beneath the structure was stabilized with vibro-flotation columns staggered on 5 foot 9 inch centers.

Owner

Nevada Department of Transportation

Project:

395 Extension (Contract EBNH-395-2(28))

Description

[16] Caltrans Historic Data 1996 through 1999

The project involved the construction of MSE walls for four bridge structures at the Brown School, South Meadows, Zolezzi Lane, and Mount Rose Interchanges. The project involved 16,000 ft^2 of wall surface with 7,800 yd^3 of granular backfill.

Owner

Nevada Department of Transportation

5.2.12 Advantages and Disadvantages of Mechanically Stabilized Earth

Advantages:

- Wall construction is relatively simple and rapid.
- Flexible and well suited for high seismic regions.
- Constructible in all seasons.
- Geocomposite inclusions resist corrosion.

Disadvantages:

- UPRR must operate on shoofly for duration of construction.
- Require significant right-of-way easement for construction and maintenance.
- Construction is bottom-up, rather than top-down, which requires slopes to be temporarily supported or laid back and dewatered during construction.
- Site soils potentially corrosive to metallic reinforcement strip, requiring geocomposites.
- Reinforced soil areas inaccessible for future utility construction or access.
- Lateral movement is required to mobilize the wall system.

5.2.13 Application

5.2.13.1 Area of Use

MSE Wall construction is suitable for most cohesionless, granular soils, which are typical of the City of Reno geology. These walls must be constructed in dry materials and do not provide a water seepage barrier. However, in Zone 1 locations in areas with wider right-of-way and no utility coordination problems, these walls are appropriate. However, large surcharge loads should be avoided to reduce the number and length of inclusions.

Since these walls are not intended to support large surcharges such as the railroad (Cooper E-18) loading, they should be avoided in the regions where the shoofly would load this wall system. Specifically, MSE walls on the south side of the trench should be avoided in the region of the Rusty Spike substation.

5.2.13.2 Installation Procedures

The installation procedure (bottom-up) for a mechanically stabilized embankment is simple and swift. Following is a brief explanation of the installation procedure and the related negative impacts in the City of Reno vicinity:

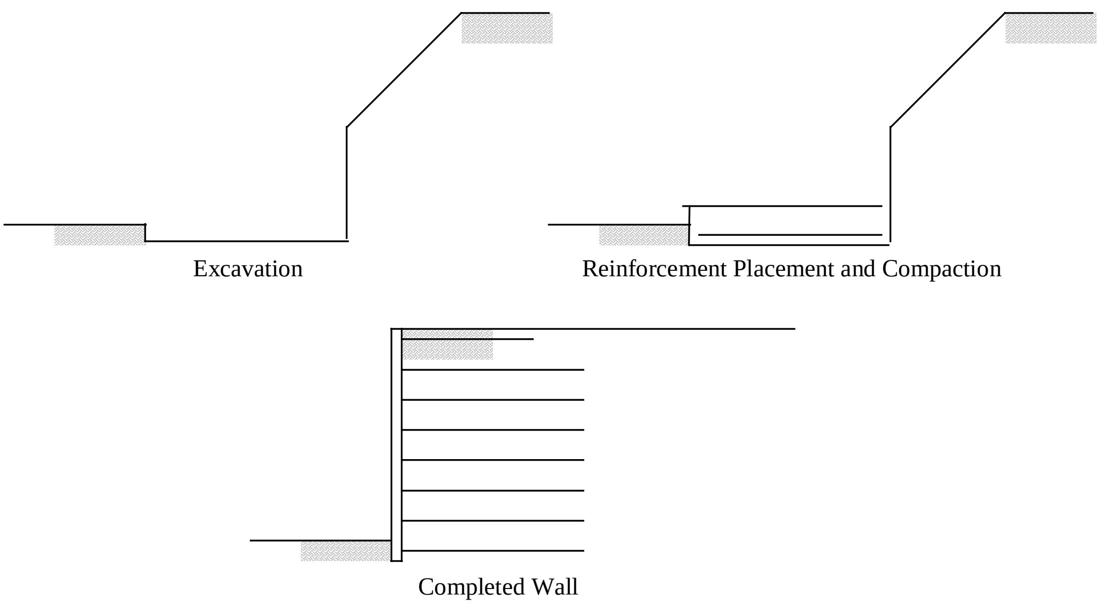
1. <u>Excavation</u>-
   Existing soil is excavated from the original ground to the bottom of the wall system. The major drawback to this form of construction is the need to execute large excavations. In addition, the soil must be laterally supported or laid back during construction.

   Since excavation procedures would encroach on the UPRR rail line, the UPRR trains would have to be moved to the shoofly for the entire construction period. Increased traffic and local business impacts would inevitably result.

2. <u>Reinforcement Placement</u>-
   Reinforcing inclusions are placed between rows of compacted fill. These inclusions are spaced approximated 2 to 3 feet vertically in lifts.

3. <u>Compaction</u>-
   Engineered fill is placed over the inclusions and compacted.

This procedure is repeated until the full wall height is constructed.



MSE Construction

5.2.13.3 Functional Performance

MSE walls have been constructed to maintain vertical, or near vertical, faces for excavations and fill for many years throughout the world. These walls pose no functionally fatal flaw that would prevent them from being used in the Reno Railroad Corridor. These walls are expected to perform extremely well.

5.2.13.4 Materials and Equipment Availability

MSE walls are constructed of simple, common materials with typical earthworking equipment. The simplistic nature of this wall type lends itself to a swift installation schedule and minimizes construction uncertainties. Therefore, based on materials and equipment; this wall is appropriate for any Zone 1 segment within the Reno Railroad Corridor.

5.2.13.5 Conclusions

Mechanically stabilized earth walls are structurally sound and promptly constructed, and utilize common materials. These walls may be recommended for those areas in Zone 1 with ample right-of-way, without utility restriction, and away from regions of large surcharges. It is suggested that this construction technique is applicable to the Reno Railroad Corridor from the western terminus to Keystone Avenue and then again from the East Side of the Rusty Spike Substation to the eastern terminus.

5.3 Micropile Walls (Zone 1)

5.3.1 Methodology

Micropiles are small-diameter, bored, grouted in-place piles constructed with some form of steel reinforcement. Micropile drilling is accomplished with a system providing holes 5- to 8-inches in diameter and drilled to a depth of approximately 1.5 times the wall height. These piles can sustain axial and/or lateral loads. Two types of micropile walls are typically constructed: 1) networks of piles (e.g., reticulated micropile walls [RMP]) and 2) groups of piles (e.g., Type "A" walls).

An RMP wall is an interlocking network of vertical and battered micropiles that "knit together" a composite, reinforced soil mass. Steel reinforcement allows the overall mass to resist compression, tension, and shear. The design concept used for an RMP wall is similar to that used for a gravity retaining structure. The RMP is designed to act as a monolithic unit; therefore, it is necessary to assume that the stresses acting on the wall are distributed to both the soil and piles.

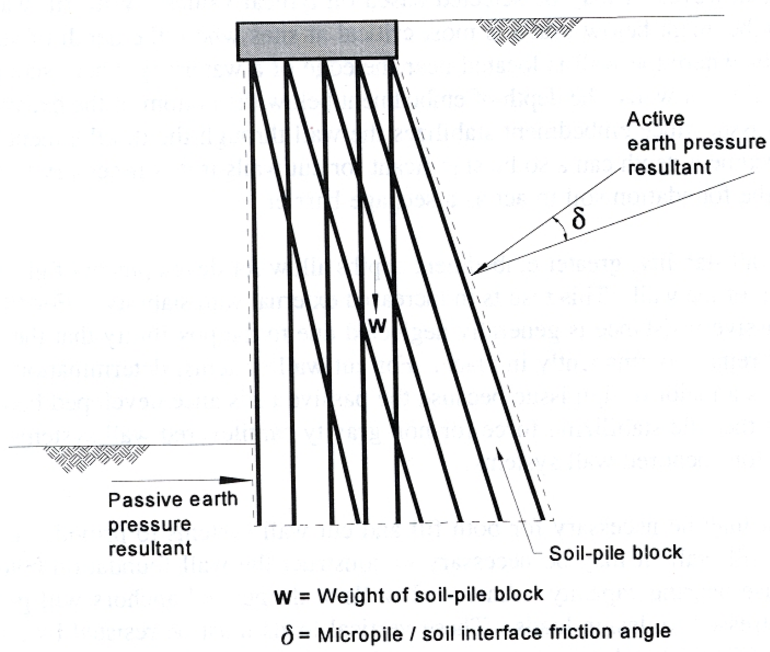


Figure 5-2 Micropiling Configuration

A Type "A" wall is used to transfer load from the surrounding soils to a greater depth. The micropile wall is constructed in an A-frame configuration so the rear legs are in tension and the front legs are in compression. Depending on the application, the rear legs may also be required to bend across the failure plane. Due to its small base, the end bearing capacity of an individual micropile is limited and the load is normally transferred through skin friction, resisting both compressive and tensile forces. The piles are installed at close spacing to retain soil between the piles.

As the excavation is performed, a facing of shotcrete, reinforced with a welded wire fabric or metal or synthetic fibers, is applied at each lift. The final facing normally consists of cast-in-place concrete or a second layer of shotcrete.

The micropiles are connected at the ground surface to a reinforced concrete cap beam, which provides additional resistance. The drilling equipment used to install micropiles is similar to the equipment used for installation of soil anchors.

5.3.2 Applicability to Soil Conditions

Micropile walls can be constructed in all types of soils. However, drilling and grouting through hard obstructions and boulders, typical of the City of Reno geology, can be time consuming and costly. Drilling progress in small diameter holes is, however, considerably faster than larger diameter holes. Installation of micropiles in openwork gravel and cobble deposits can result in excessive grout loss. A micropile wall is not appropriate in excavations below the groundwater table without the prior construction of a cut-off wall.

5.3.3 Stability of Wall Construction

Micropiling is accomplished by installing piling in the existing soil. No excavation is required for installation of this technique. In addition, the slope will be stabilized by the piles during the exposure of the wall surface, thereby eliminating the risk of instability due to shoofly operations.

5.3.4 Abutment Related Issues

Micropiles take the vertical structural load directly (Case 1) or create a reinforced mass (Case 2) whereupon the load is taken by the mass, and not the piles directly.

For Case 1, the superstructure loads are transferred to the soil via the pile system, much like a traditional deep foundation.

For Case 2, the internal shear capacity of the soil mass is increased enough to resist the forces imposed by brides. With either case, a cast-in-place abutment structure will be required.

With either design, micropiles will be able to support bridge structures. Therefore, based on their function at bridge abutments, they are recommend in Zone 1.

5.3.5 Duration of Construction

Since small diameter shafts would have to be drilled in the City of Reno stratum, a long construction schedule would be required. With the relatively small spacing (approximately 2 feet) between micropiles, applying this soil improvement technique over a large distance of wall would be infeasible. The required time to construct 10 linear feet of wall is 2.5 days (58 ft^2/shift)[17]. Due to scheduling limitations, the application of micropiles would have to be confined to localized regions at the City of Reno project site.

[17] Donald A. Bruce, PhD, C. Eng, FICE, GEOSYSTEMS, LP, Venetia PA

5.3.6 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Railroad Corridor.

Two major factors affect noise and traffic impacts from construction of wall systems in the Reno Railroad Corridor: 1) equipment use and 2) duration of construction.

The following is a brief list of the major construction equipment used to construct slurry-diaphragm walls:

- Drill Rig
- Grout pump
- Grout batch plant
- Materials delivery trucks
- Crane
- Front loader

5.3.6.1 Noise Impacts

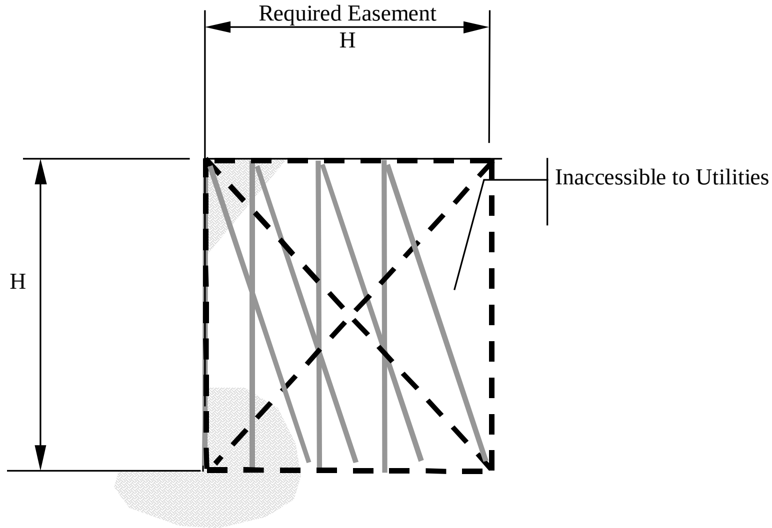
The vehicles in the list above are common for construction sites. Although not quiet, they introduce only minor levels of noise above average daily traffic, and create less noise than is produced by train whistles in the current rail configuration. Overall, this construction method, if used in localized applications, produces only minimal noise impacts.

5.3.6.2 Traffic Impacts

Micropile installation requires close operation of multiple vehicles. Operation outside the trench limits is required to install the micropiles on the existing rail profile. The top surface of these walls would require the majority of the piling and it is anticipated that the micropiling effort would require closing of adjacent local streets running parallel to the alignment.

5.3.7 Right-Of-Way Impact

Micropile reinforced slopes have the potential to create larger than average right-of-way impacts. Similar to the construction of MSE walls, micropiles require the installation of inclusions in the slope. However, unlike MSE walls, vertical slopes are not possible with this soil improvement technique. Therefore, the impacts from micropile walls are greater than MSE construction. The required easement is typically the height of the wall. For an application of micropiling in Zone 1, this impact may be as great as 15 feet or more. As seen in the diagram below, the region behind the wall face is inaccessible to local utilities or other underground improvements. The micropile technique, although a functional retaining system, is only recommended for locations along the alignment where right-of-way is plentiful and utility conflicts are minimal. It is suggested that these walls apply to the same geographic location as MSE construction, specifically, in the regions between 1) the western trench terminus and Keystone Avenue and 2) Lake Street and the eastern trench terminus.



Micropile Right-of-Way Impacts

5.3.8 Aesthetics

Micropile reinforced soil is finished with a reinforced shotcreted surface. The finished appearance is clean and neat. Additional aesthetic treatments are available for shotcreted blankets. Possibilities include color, texture, or feature lines. These walls may also be vegetated if the shotcrete is eliminated. . Adding aesthetic treatments to the shotcrete surface may double the required construction time for finish work. This equates to a 20 percent increase in the schedule for soil mass reinforcement and could add approximately $5 to $15 per square foot of treated surface.

5.3.9 Conceptual Calculations

Examining the soil mass as a gravity retaining wall, the required width would be approximately double the design height (e.g., a wall with a design height of 10 feet would require a soil mass approximately 20 feet wide). The soil mass created with these micropiles would be capable of resisting overturning and sliding due to active soil pressure. Additionally, these walls may require more width, or deeper piles, to counteract the forces due to bridge loads.

5.3.10 Cost

Based on historic data of micropile construction, it is estimated that the cost of installation is approximately $100 per linear foot of piling. Based on wall height and on the design requirements of two to three foot spacing and an average length of 14 to 18 feet, the estimated cost per square foot of wall surface is $67[18]. This cost is on the order of 10 times more expensive than traditional soil retention systems. For this reason, micropiling is not recommended for the Reno Railroad Corridor project.

[18] Ron Chapman, Schnabel Foundation Company, Walnut Creek, CA

5.3.11 History of Successful Application

The following section contains a list of similar successful projects using the micropiling technique:

Project

State Route 4023, Armstrong County, Pennsylvania

Description:

The project involved stabilizing 250 feet of two-lane roadway and railroad track damaged by slope movement towards the Allegheny River. Subsurface conditions consisted of random fill, stiff colluvial clay with rock fragments on weathered claystone, and competent sandstone.

The repair included an "A" frame insert wall made up of four rows of 6-inch-diameter pin piles extending across the slip plane and into competent rock. Micropile depths varied from 23 to 35 feet.

Owner

Pennsylvania Department of Transportation (PADOT)

Project:

Blue Heron Road, Big South Fork River, Kentucky

Description:

An "A" frame micropile wall was used to stop a moving slope downhill from a bridge abutment and above a land pier supporting a historic railroad bridge. Subsurface conditions consisted of medium stiff to stiff clay and shell bedrock. A back analysis of the wall indicated that the creeping slope had a total driving load of approximately 45-kips/linear foot. Micropiles were advanced to a depth of 25 feet below existing grade.

Owner

U.S. Army Corps of Engineers

5.3.12 Advantages and Disadvantages of Micropiles

Advantages:

- Constructible in low headroom areas, such as under bridge decks.
- Relatively small and lightweight equipment required for installation.
- Adaptable to many different types of soil conditions.
- Specialty contractor and designer required.

Disadvantages:

- Not appropriate below the groundwater table without the construction of a cut-off wall.
- Significant right-of-way easements required.
- Design procedures are not well established.
- The system requires more drilling than other methods.
- Requires permanent facing wall.

5.3.13 Application

5.3.13.1 Area of Use

Micropiles, used for stabilizing slopes and supporting both vertical and lateral loads, are suitable for many soil types. However, the suitability of micropiles may be limited due to right-of-way encroachment and groundwater infiltration prevention issues.

Micropile-reinforced soil masses affect adjacent rights-of-way by eliminating the possibility of installing underground improvements, including utilities, and requiring a permanent maintenance easement over the region of application. For these reasons, this technique is recommended in localized regions that have greater than twice the wall height in right-of-way beyond the theoretical wall face.

5.3.13.2 Installation Procedures

The installation procedures for micropiles preclude them from use for long distances along the trench alignment. This retaining system requires a larger number of drilled holes than any other retaining options examined in this report. The vast number of holes, in combination with boulder inclusive soils, negates long stretches of application. However, this technique may be applied in smaller locations such as at bridge abutments or near the ends of the trench, where less drilling is required.

To counteract the associated higher costs of micropiling, this is a top-down procedure and eliminates the need for soil handling during construction of the reinforced soil embankments.

5.3.13.3 Functional Performance

Micropiling offers an effective and efficient means of stabilizing soil in a near vertical configuration and underpinning structures. Using this construction method would result in a successful depression of the Union Pacific Railroad tracks in the Reno Railroad Corridor.

5.3.13.4 Materials and Equipment Availability

Micropiling requires specialized equipment and a specialty contractor. These requirements, while reducing actual costs when compared to a general contractor's performance, are not enough to keep the costs within reasonable limits. The time consuming and labor intensive nature of this technique, combined with the requirement to drill an exorbitant number of holes, makes this choice cost prohibitive.

5.3.13.5 Conclusions

Based on functional performance, this wall system could be used effectively for various regions throughout the Reno Railroad Corridor. However, the high costs and slow production rates of this option eliminates it from any recommendation.

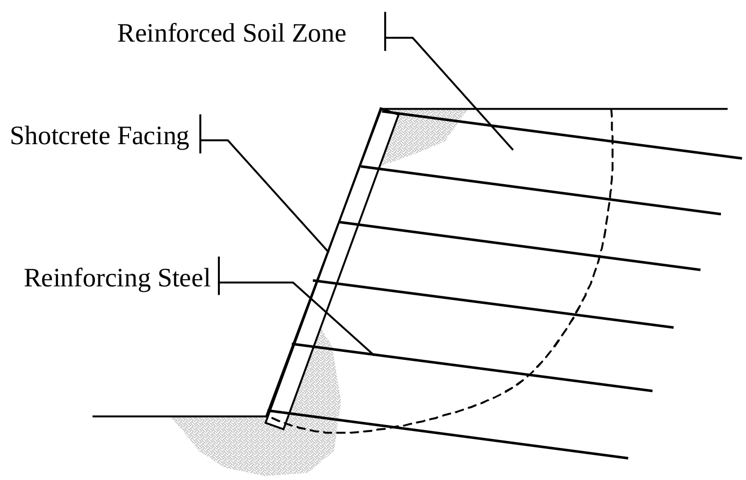
5.4 Soil Nail Walls (Zone 1)

5.4.1 Methodology

Installed through the face of an excavation at an inclination of approximately 15 degrees downward from horizontal, soil nail walls consist of closely spaced steel bar reinforcements. The soil nails reinforce the in-situ soil mass and allow for top-down construction of an excavation. The excavation is typically advanced in 5-foot lifts with soil nails installed on 5-foot centers. The nails increase the shear strength of the overall soil mass and limit displacement during and after excavation

Soil nails are commonly referred to as "passive" inclusions. The nails are not pre-tensioned but are forced into tension as the surrounding ground deforms laterally in response to the loss of support caused by continued excavation. In order to activate the reinforcing nails, deformations of between 0.1 and 0.3 percent of the wall height should be expected.

Soil nails are typically grouted in 8-inch diameter pre-drilled holes in the United States. The face of the excavation is shotcreted to create a wall surface to provide short-term cohesion. If necessary, a permanent wall surface can be attached to the shotcrete surface.



Soil Nailing

5.4.2 Applicability to Soil Conditions

Soil nail walls can be constructed in many types of soil. However, these walls are not suitable for loose granular soils, uniformly graded granular soils, organic soils, and soft or highly plastic clays. Installing nails through openwork gravel and cobble deposits can result in excessive grout loss. Due to the presence of cobble in the Reno geology, it is anticipated that the nails will be grouted in pre-drilled holes.

The City of Reno geology consists of cobbles, gravels, sand, and fines. The presence of the fines in the subsurface conditions allows this option to be feasible. However, since non-cohesive constituents comprise a large portion of the soil in the City of Reno vicinity, the face of soil-nailed slopes is not stable if kept vertical and therefore must be laid back.

Furthermore, soil nailing cannot be installed below the groundwater level without dewatering and the final wall is not watertight; hence it has limited effectiveness for the central portion of the proposed project. Soil nails usually extend 50 to 100 percent of the wall height behind the wall facing.

5.4.3 Stability of Wall Construction

Soil nails are installed as the wall face is being exposed through excavation procedures in the trench. The installation of these inclusions fortifies the slope and adds to the stability of the wall system. However, the magnitude of surcharge imposed on a soil nailed slope due to heavy rail operation is in excess of practical limits. Therefore, supplemental slope stability techniques may be required to ensure slope stability during construction adjacent to live rail.

5.4.4 Abutment Related Issues

Soil-nailed systems increase the internal shear capacity of the soil, thus helping support vertical and horizontal loads. However, the amount of increased capacity in internal shear strength of the soil mass is minimal compared to the added loads imposed by the abutment structures. Additional reinforcement is required, through a deep foundation at the abutment or through installation of post-tensioned grouted ground anchors (tiebacks).

5.4.5 Duration of Construction

Examining the preliminary geotechnical report and understanding the soil conditions of the Reno Railroad Corridor is important in estimating the amount of time needed for construction of any wall system. For the purposes of this report, and analyzing soil nailing, it is assumed that each nail would be grouted in pre-drilled holes. Given these assumptions, the anticipated production rate of soil nailing in the Reno vicinity is 1,000 ft^2/shift[19]. This progress is swift and compares well to the required time for traditional reinforced concrete cantilever retaining walls.

5.4.6 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Corridor.

Two major factors affect noise and traffic impacts from construction of wall systems in the Reno Corridor: 1) equipment use and 2) duration of construction.

The following is a brief list of the major construction equipment used to construct soil nailed walls:

- Drill Rig
- Grout pump
- Grout delivery trucks (Optional)
- Materials delivery trucks

[19] Donald A. Bruce, PhD, C. Eng, FICE, GEOSYSTEMS, LP, Venetia PA

- Crane
- Front loader
- Sheepsfoot compactor
- Scraper
- Shotcrete Pump

5.4.6.1 Noise Impacts

The vehicles in the above list are common for construction sites. Although not quiet, they introduce only minor levels of noise above average daily traffic, and create less noise than is produced by train whistles in the current rail configuration. Overall, this construction method, if used in localized applications, produces only minimal noise impacts.

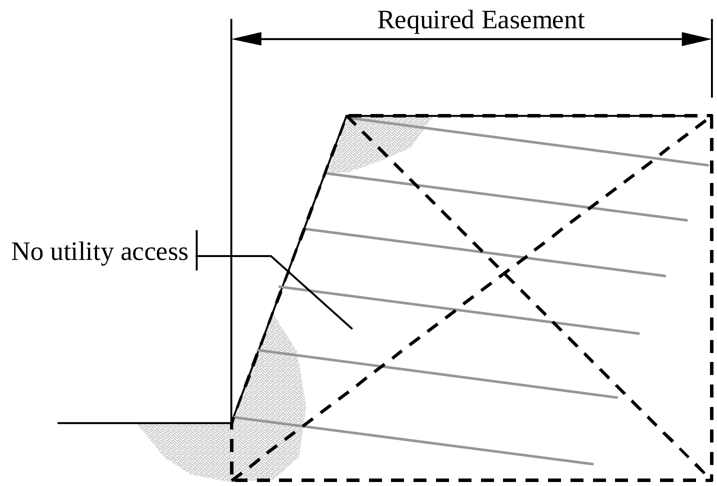
5.4.6.2 Traffic Impacts

Top-down construction, combined with installation techniques that allow the construction of soil-nailed wall systems within the trench limits, favors this option for the walls in Zone 1. However, the installation of the inclusions for these walls may not be accomplished while the UPRR is operating on the opposite rail. The traffic implications of soil nails are similar to those of the other bottom-up options, namely MSE and cantilever walls.

5.4.7 Right-Of-Way Impact

Since soil nails use long reinforcing bars installed into the side of the embankment, right-of-way impacts to local utilities and property owners that are adjacent to the soil nailed earth-retaining system would result. The magnitude of impact is directly related to the length of the reinforcing bars. At this time, the length of the reinforcing steel is assumed to be approximately 60 to 70 percent of design height of the wall (e.g., a design height of 10 feet would require the reinforcement to extend 6- to 10-feet beyond the theoretical face of the abutment).

In addition, it is not practical to create a vertical face for this wall system. Therefore, as the excavation is laid back (approximately 15 degrees from vertical), the required underground easement is increased. This impact is minimal for shorter walls, but the required increase to the underground easement is approximately 6 feet for a design height of 22 feet.



Soil Nailing

Based on the *Draft Reno Railroad Corridor Environmental Impact Statement and the Means* and *Methods Analysis Report*, in the region of proposed application of soil nailing (Zone 1), adequate underground easements exist. However, with taller walls requiring more right-of-way than vertical face techniques (cantilever walls or mechanically stabilized earth), these walls may be precluded from use in the Reno Railroad Corridor as the alignment approaches Zone 2. In addition, the presence of a proposed utility corridor along the north trench wall would eliminate soil nailing from contention.

5.4.8 Aesthetics

Soil nail reinforced walls are finished with a reinforced shotcreted surface. The finished appearance is clean and neat. Additional aesthetic treatments are available for shotcreted blankets. Possibilities include color, texture, or feature lines. Aesthetic treatments may be added to the shotcreted surface but may be expected to double the construction time needed for finish work, The schedule would be extended by approximately 20 percent and the cost increased by $5 to $15 per square foot by use of this treatment.

5.4.9 Conceptual Calculations

Based on soil parameters that include lateral pressure diagrams appropriate for sandy, granular material, an internal angle of friction of 34° (NDOT/FHWA requirement), and a dry unit weight of 115 pcf, the following table was developed for a wall with the design height of 22 feet (maximum condition). This table indicates the level number, distance from the top of the trench to the inclusion (z), and the required length for the inclusion.

Level	z	Length.
1	5 feet	19.0 feet
2	10 feet	19.0 feet
3	15 feet	12.0 feet
4	20 feet	10.0 feet

Based on the calculation summary above, caution should be taken when installing the first 2 rows of reinforcement, as these are the most likely to result in impacts to adjacent structures and utilities.

5.4.10 Cost[20]

Driven or fired soil nailing applications are not applicable to the City of Reno vicinity. However, in the City of Reno vicinity, these inclusions must be grouted into predrilled holes. An estimated cost of $25/ft^2 is reasonable for the City of Reno geology. This cost includes the reinforcement, grouting and shot-creted facing.

5.4.11 History of Successful Application

The following are successful soil-nailed projects that produced results similar to those desired in the Reno Railroad Corridor.

Project:

Project C, Silver Legacy Casino, Reno, Nevada

Description:

The project required approximately 34,000 square feet of shoring in two blocks of downtown Reno to depths of up to 30 feet and construction of a tunnel between the hotel and casino. Soil conditions consisted of dense glacial outwash materials of very dense sands, gravel, cobbles, and boulders. The shoring system consisted of battered soil nailing and shotcrete. To install the soil nails in caving, noncohesive to low-cohesive soils, the nails were constructed to within about 3 feet from the final wall line using a self-grouting injection anchor system. Requiring an accelerated construction schedule, the project successfully completed in 7 weeks.

Owner

Circus-Circus/El Dorado Hotel Joint Venture

Specialty Contractor

Schnabel Foundation Company

Project:

Second Street Junction Structure, Phoenix, Arizona

Description:

The 62-foot-deep excavation for the Second Street Junction Structure was made entirely in sand, gravel, and cobbles. The top 25 feet of the excavation was sloped at a batter of approximately 1:1 (H:V). The bottom

[20] Ron Chapman, Schnabel Foundation Company, Walnut Creek, CA

37 feet was excavated nearly vertically, supported by six rows of soil nails and a 4-inch-thick shotcrete face.

Owner

Arizona Department of Transportation

General Contractor

Tanner

Project:

The Grace Cathedral, San Francisco, California

Description:

Construction of the alterations and improvements to historic Grace Cathedral required excavations up to 45 feet deep along the edge of an existing school, the Cathedral, and a historic fence along Sacramento Street in the Nob Hill district. A total of 12,000 square feet of wall-line soil-nailing and shotcrete shoring were installed to resist lateral earth pressures. Slant pile underpinning was installed underneath the school foundations. The historic fence and Cathedral bell tower were underpinned with micropiles. Recorded horizontal movements were less than 0.24 inches at the deepest section under the school foundation.

Owner

The Grace Cathedral

Contractor

Swinerton & Walberg

Specialty Contractor

Schnabel Foundation Company

5.4.12 Advantages and Disadvantages of Soil Nailing

Advantages:

- Constructible in low headroom areas, such as under bridge decks.
- Relatively small and lightweight equipment required for installation.
- Eliminate the need for internal bracing so unobstructed workspace is achieved.
- Greater redundancies due to relatively close spacing of reinforcing elements.
- Adaptable to varying site conditions and wall alignments.

Disadvantages:

- Significant right-of-way easements required for installation of soil nails.

- Construction below groundwater level requires that excavation face be dewatered during construction.
- Potential interference with existing utilities and structures.
- Some movement is required in the wall to fully mobilize the wall system.
- Normally secondary construction of a permanent wall is required. Permanent soil nail walls are possible, however, provided there are permanent easements behind the wall and suitable corrosion protection is provided.

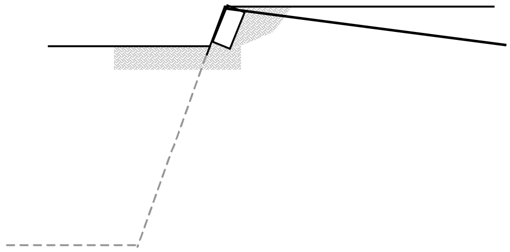
5.4.13 Application

5.4.13.1 Area of Use

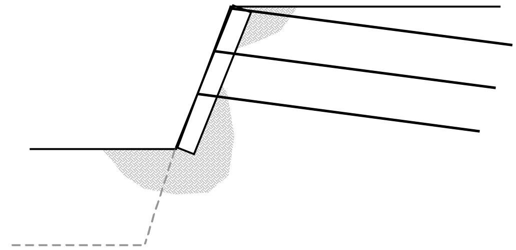
Soil nail-reinforced soil masses provide increased internal shear capacity. This increased shear capacity, combined with top-down construction; makes soil nailing an attractive reinforced earth option. This technique is applicable in the regions where the interior wall face is not required to be vertical and sufficient right-of-way is available (toward the termini of the trench). However, the major limitations with soil nailing are its ability to encroach on adjacent utilities and eliminate the future potential for underground infrastructure improvements. Areas in which these issues are a consideration must be avoided with soil nail reinforcing.

5.4.13.2 Installation Procedures

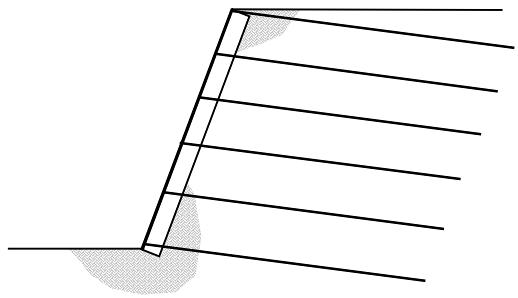
The installation procedure is top-down and may occur in restricted access locations, such as adjacent or under existing structures, or low head room applications. As seen below, the installation procedure includes a single lift of soil to excavate before soil nail installation. This process is repeated until the entire excavation is completed. Supporting near vertical (approximately 15° to 20° downward from vertical) faces during excavation makes this procedure beneficial for shorter walls used in the Reno Railroad Corridor. The 15° to 20° angles become detrimental to the applicability of taller walls. With taller walls, larger easements are required.



Step 1: Excavated Lift, Shot-crete and Nail Installation



Step 2: Continued Cycle



Step 3: Completed Construction

5.4.13.3 Functional Performance

Functionally, these soil masses are adequate in regions of low or no surcharge loading. The increase in internal shear capacity is less than for other systems of inclusive reinforcement. Additionally, these soil masses are not intended to support bridge loads. Therefore, based on its functional performance, soil nailing has applications on the shorter walls at the ends of the trench, particularly in locations without surcharge loads.

5.4.13.4 Materials and Equipment Availability

The material used for soil nails (steel reinforcing bars), shot-crete, welded-wire fabric, and drainage strips are readily available in the City of Reno vicinity. The equipment, although specialized, is also available. The availability of both materials and equipment makes this a viable option in the Reno Railroad Corridor.

5.4.13.5 Conclusions

Based on functional performance, installation practices, areas of use, and compatibility with the existing Reno geology, this soil reinforcement technique is a viable option for short walls, in localized applications where no surcharges and ample right-of-way (including required batter) are present. Therefore, these walls are recommended for Zone 1 in regions that have wall heights less than 8 feet, where the impact to easements is within 2 feet of the competing options (cantilever walls and mechanically stabilized earth).

5.5 Soldier Piles and Lagging (Zone 1)

5.5.1 Methodology

Figure 5-3 Soldier Piles with Timber Laggin

Typically used for temporary bracing of large excavations, soldier pile and lagging systems consist of vertical, steel rolled sections installed every 5 feet to 10 feet along the perimeter of an excavation with timber or precast concrete lagging spanning the distance between the piles. The steel soldier piles can either be driven into place with impact or vibratory hammers, or set into predrilled holes generally filled with concrete below the invert of the excavation and with lean concrete above the invert. Predrilled holes are typically utilized for dense soil conditions and in areas where it is required to minimize construction vibrations. As the excavation proceeds downward, horizontal timber or precast concrete lagging is installed behind the steel flanges of the vertical piles. Soldier pile walls can be internally braced with struts or tied back with anchors. Cantilevered soldier pile and lagging walls are normally effective to a height of approximately 15 feet. Deeper excavations are possible provided the wall system is braced or anchored.

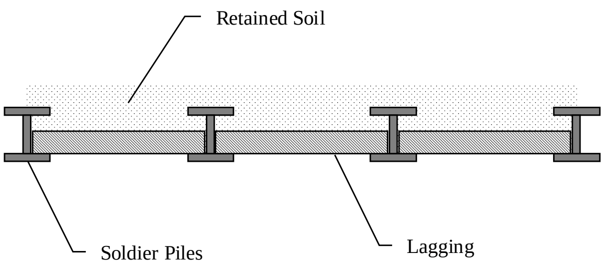


Figure 5-4 Partial Plan

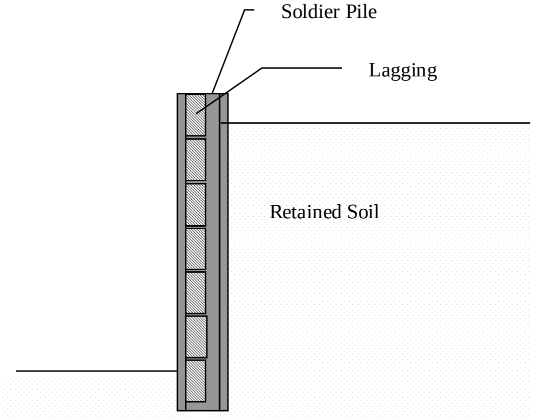


Figure 5-5 Typical Section

5.5.2 Applicability to Soil Conditions

Soldier pile and lagging systems are generally used for excavation support of cohesive soils and granular soils above the groundwater table. Soldier pile and lagging systems are not suitable for granular soils below the groundwater because it is difficult to install lagging without experiencing loss of ground behind the support wall. This loss of ground can result in potentially large ground movements outside of the excavation limits. However, in the region of soldier pile installation, these concerns can be mitigated with grouting below the groundwater table. In addition, since soldier pile and lagging walls are highly pervious, excavations below the groundwater table require dewatering measures that result in groundwater drawdown outside of the excavation limits.

Driven soldier piles are probably not feasible due to the fact that boulders and dense coarse soils cause frequent refusal. Drilled soldier piles are feasible above the water table; however, drilling progress is likely to be slow due to difficulties with cobbles and boulders. Drilling equipment utilizing casing rotators or down-hole hammers may increase drilling rates over those experienced in the 39-inch-diameter exploratory holes. Still larger-diameter holes may better deal with the boulders than the test holes.

The other subsurface condition to be addressed is corrosivity of steel elements in native soils. Subsurface investigations conducted by Kleinfelder indicate a single soil sample at a depth of 6.5 feet to have a high potential for corrosion of steel, leading Kleinfelder to recommend coating or otherwise protecting metal inclusions. This protection is recommended for the steel sections inserted in the pre-drilled shafts.

5.5.3 Stability of Wall Construction

Special geotechnical consideration is required for soldier pile and lagging walls. In the proposed methodology, excavations required for pile installation may be held open using bentonite slurry until they can be backfilled with soil or lean concrete mix. The outward pressure provided by the slurry, on the sides of the excavation, resists the lateral forces that cause caving. However, a major concern of applying this technique to the excavation in the Reno Railroad Corridor is the stability of this system when employed adjacent to live heavy rail.

Based on the analysis that was performed on the slurry-diaphragm wall technique, having a relatively larger open excavation, this method shows little risk of collapse or unacceptable vertical deflections.

5.5.4 Abutment Related Issues

Soldier piles have been successfully used to resist both vertical and horizontal loads. The lateral resistance may be accomplished through deeper piles, larger pile sections, closer pile spacing, grouted ground anchors or “dead men.” The flexibility of this system lends itself to the use of shallow abutment foundations that influence the wall structure with additional vertical and lateral loads.

Alternatively, the soldier pile and lagging configurations may be modified to allow for simple installation of high-cantilever, traditionally reinforced concrete abutment walls. Since soldier pile walls are not intended to constrain groundwater, the continuity between a reinforced concrete wall and soldier piles and lagging is not crucial.

5.5.5 Duration of Construction

Soldier pile and lagging is a top-down solution requiring drilled shafts to install the soldier pile structural elements. Lagging is installed as the excavation progresses. The fact that drilling would be required in the City of Reno slows the construction of this wall system. Based on findings in the initial geotechnical investigation, it is estimated that the rate of production for drilling these shafts is approximately 13 feet per day. Based on a center-to-center spacing of 10 feet and an average wall height of approximately 8 feet, 10 to 15 linear feet of wall could be constructed per day for each crew at each heading (215 ft^2/shift)[21]. These production rate estimates do not include the mass excavation and installation of lagging and grouted anchors that are required.

5.5.6 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Corridor.

Two major factors affect noise and traffic impacts from construction of wall systems in the Reno Corridor: 1) equipment use and 2) duration of construction.

[21] Mark Doehring, Kleinfelder, Reno, NV

The following is a brief list of the major construction equipment used to construct soldier pile and lagging walls:

- Drill Rig
- Materials delivery trucks
- Crane
- Spoils Hauling Truck

5.5.6.1 Noise Impacts

The vehicles listed above are common for construction sites. Although not quiet, they introduce only minor levels of noise above average daily traffic, and create less noise than is produced by train whistles in the current rail configuration.

An additional consideration of project duration is required at this stage of analysis. The length of construction, based on the region of application, may be only a few days to years. The entire trench construction project is intended to be completed with 24 months, thereby leaving this specific construction equipment on-site for as much as 3 construction seasons, providing for breaks for inclement weather. Overall, the equipment used to construct soldier pile and lagging walls and the time frame during which the equipment would be used are expected to result in only minimal noise impacts.

5.5.6.2 Traffic Impacts

As demonstrated for secant/tangent pile walls, soldier pile and lagging walls can be constructed while the UPRR is in operation on its main alignment. This, combined with the ability to perform the construction within the trench limits, reduces the impacts to local traffic. For this reason, soldier pile and lagging construction is recommended for any location within Zone 1.

5.5.7 Right-Of-Way Impact

Based on grouted ground anchor designs, and *the Draft Reno Railroad Corridor Environmental Impact Statement*, the maximum impact to adjacent underground easements is approximately 34 feet, which is well within the right-of-way limits, as examined in the DEIS. Therefore, soldier pile and lagging wall systems do not result in detrimental right-of-way impacts that would prevent them from being constructed within the proposed project alignment.

5.5.8 Aesthetics

Among all vertical face choices in this document, soldier pile walls are the least flexible in regard to aesthetic treatments. The exposed wall surface is comprised of vertical steel members and timber or reinforced concrete cross-bracing. Applying facing treatments requires expensive detailing and labor intensive procedures. Alternatively, these walls systems may be shotcreted and trowel finished. Shotcreting, even though expensive (approximately $10 per square foot of face) may be treated similarly to reinforced concrete retaining walls. These treatments include paint, color impregnation, texturing, or application of architectural feature lines. The addition of these treatments would add approximately 20% to the construction time. Therefore, unless necessary, it is not recommended that aesthetic treatments be applied to soldier pile and lagging walls.

5.5.9 Conceptual Calculations

Based on techniques used in trenching and shoring, and examining the maximum wall height in Zone 1, it was determined that pile sections (W14x159) would be required at 8-foot spacing on center and that the lagging required would be rough sawn 4x12 lumber, or equivalent. Based on the analysis, the extension of the pile below the plug is required to be 33.5 feet, which is much larger than that of secant/tangent piles or slurry-diaphragm walls. This is based on the assumption that the passive resistance of the plug in the localized area of the pile is equivalent to that of the surrounding soil. After plug design, these values can be reduced by a detailed examination of the performance of the piles. Currently, no plug option or performance criterion is available for further consideration. In addition, the extension of the piles and pile sections may be reduced when using a secondary lateral support system (not examined for this report). Based on preliminary engineering soldier piling is feasible and efficient for construction of supported excavations in the Reno Railroad Corridor.

5.5.10 Cost[22]

Historical data indicate that the cost of soldier pile and lagging walls is approximately $25 per square foot of excavated wall face. The addition of struts or grouted ground anchors is not included in this estimate. An estimated cost of $3,000, and $7,500 per tieback anchor or strut should be assumed, respectfully.

5.5.11 History of Successful Application

Historically, cases of the successful application of soldier pile and lagging walls are too numerous to list. This construction technique has been used to successfully brace excavations and construct permanent structural walls throughout the world for over 100 years.

5.5.12 Advantages and Disadvantages of Soldier Pile and Lagging Walls

Advantages:

- Cost effective, conventional wall system with well-established design procedures.
- Individual solider pile sections are less likely to encounter boulder obstructions than continuous steel sheet piles.
- Can be drilled through dense soils and obstructions.
- Used for shallow excavations above the groundwater table.

Disadvantages:

- Predrilling through hard soils and boulder obstructions can be a significant cost component.

[22] Caltrans Historic Data 1996 through 1999

- Excavation for placement of lagging between soldier piles can cause ground loss outside of excavations.
- A soldier pile and lagging wall system, if not properly braced or tied back, may undergo relatively large lateral movements that could be detrimental to adjacent structures and utilities.
- A soldier pile and lagging wall is a temporary system requiring secondary construction of a permanent wall.

5.5.13 Application

5.5.13.1 Area of Use

A strong lateral support system combined with top-down construction and minimal lateral supports for short configurations suggests the use of soldier pile and lagging walls for relatively shallow excavations at the ends of the trench system in Zone 1. Additionally, these walls, although pervious to water, are faster and more economical that most other permanent wall systems (secant/tangent walls, permeation grouting, jet grouting, Stresswall system, and micropiles).

5.5.13.2 Installation Procedures

The installation procedures, as depicted below, are top-down construction steps that can be summarized as follows:

1. Predrilled shafts-
   These shafts are predrilled along the alignment in preparation of steel pile installation. This procedure has the highest degree of uncertainty with regard to schedule, which directly affects construction costs.
2. Install Piles-
   The steel sections are installed into the shafts and grouted in at the bottom. The grout minimizes the drilled length beyond the theoretical point of fixity.
3. Excavation-
   The main trench section adjacent to the wall is excavated and lagging is inserted between the vertical piles. This process continues until the bottom of the trench is exposed. Tieback anchors are installed as the excavation progresses in deep excavations, or where additional lateral support is required.

This top-down technique allows the UPRR to maintain traffic on the opposite rail line while construction of the wall is completed. Shoofly operation is minimized with these top-down options.

5.5.13.3 Functional Performance

Functionally, soldier pile and lagging walls have performed extremely well throughout their many years of use for deep excavations and formation of permanent structural walls. If used in the Reno Railroad Corridor, it is anticipated that these walls would perform as well as any other permanent wall, vertical face option. However, since these walls are pervious, their use is restricted to Zone 1. Alternatively, these walls may be designed to include a grouted section behind the wall for applications within Zone 2.

5.5.13.4 Materials and Equipment Availability

The materials, namely steel structural sections and timber or precast concrete lagging, are readily available for use in the City of Reno vicinity. Additionally, the equipment used for installation is common and available for transportation to the City of Reno project site. The availability of both materials and installation equipment contribute to making this wall system a viable choice for the Reno Railroad Corridor.

5.5.13.5 Conclusions

Based on compatibility with the geology, applicability to the project, and a strong successful history of use for similar projects, soldier pile and lagging systems are a structural leader among the other options in Zone 1. Other than production rates, this technique compares well to cantilever walls, mechanically stabilized earth and soil nailing. Therefore, it is included in the list of candidates for construction in the Reno Railroad Corridor.

5.6 Stresswall System (Zone 1)

5.6.1 Methodology

Stresswall, an example of a proprietary wall system (many others are available), is comprised of two precast concrete elements: 1) louvered columns supported on concrete filled shafts and 2) wall panels set between the precast columns and held in place by soil pressure. The system is very similar to soldier pile and lagging with top down construction, requiring excavation to install lagging.

Numerous wall geometries and architectural styles are possible using the prefabricated components. Walls may be terraced for landscaping or installed vertically as required.

The system can be used in combination with soil anchors or struts to resist horizontal earth pressures. A typical Stresswall section is shown in Figure 5-6.

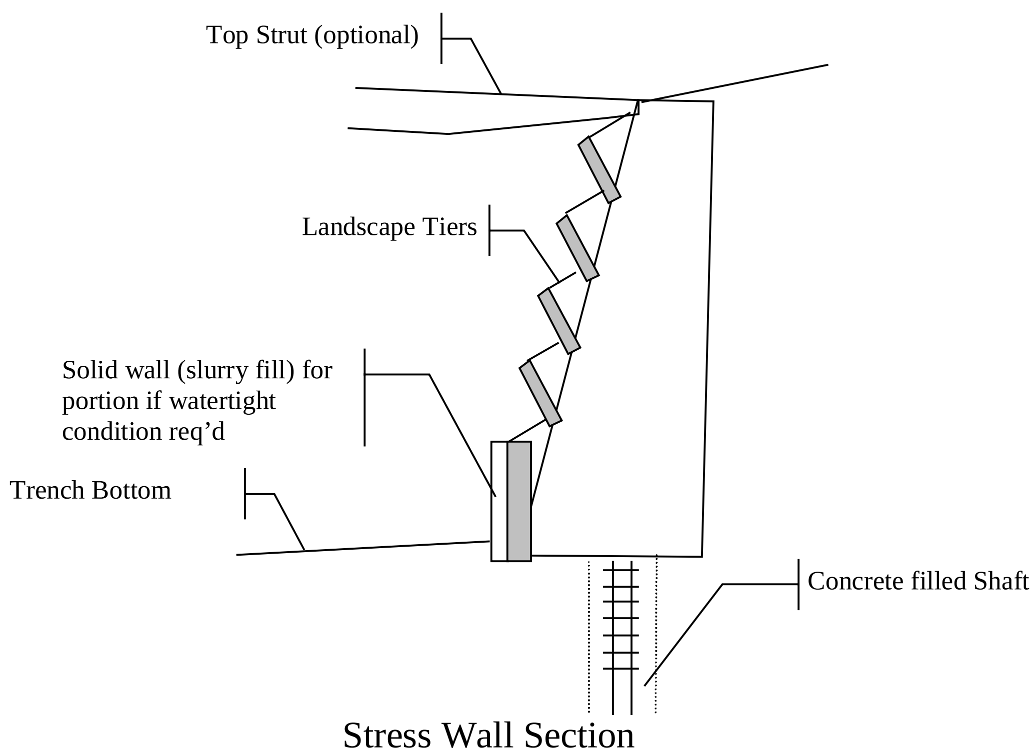


Figure 5-6 StressWall Section

5.6.2 Applicability to Soil Conditions

The method allows for top down construction, the panels being supported by a pre-cast louver column on a reinforced concrete-filled shaft. However, to install the pre-cast louver columns would require numerous large diameter borings up to 5 feet in diameter. Such excavations would be slow and costly in the dense glacial outwash soils. However, construction rates of larger diameter shafts would be faster due to the relative diameter of the inclusive cobbles. Excavations below the groundwater table require a cutoff wall or dewatering measures.

5.6.3 Abutment Related Issues

Stresswalls support the vertical face of the soil and increase the internal shear capacity of the soil, thus helping to support vertical and horizontal loads. The amount of increased capacity in internal shear strength of the soil mass is determined by wall design and soil properties. A deep foundation at the abutment or installation of post-tensioned grouted ground anchors (tiebacks) provides additional reinforcement. Stresswalls, with proper engineering, are anticipated to function adequately at the bridge abutment locations.

5.6.4 Duration of Construction

Since large diameter shafts are typically faster to drill, in cobbled soils, than medium diameter shafts, the construction of these walls systems is anticipated to be similar to the construction rates of soldier pile and lagging walls, or approximately 10 to 15 linear feet of wall (in Zone 1) per day per crew at each heading (215 ft^2/shift)[23]. Likewise, the construction process can be accelerated by working with 2 crews at each heading, thus approaching a production rate of approximately 40 to 60 linear feet of wall per day.

5.6.5 Traffic and Noise Impact

Both traffic and noise impacts must be discussed separately for the construction project in the Reno Corridor.

Two major factors affect noise and traffic impacts from construction of wall systems in the Reno Corridor: 1) equipment use and 2) duration of construction.

The following is a brief list of the major construction equipment used to construct Stresswall systems:

- Drill Rig
- Materials delivery trucks
- Crane
- Spoils Hauling Truck

5.6.5.1 Noise Impacts

The vehicles listed above are common for construction sites. Although not quiet, they introduce only minor levels of noise above average daily traffic, and create less noise than is produced by train whistles in the current rail configuration.

An additional consideration of project duration is required at this stage of analysis. The length of construction, based on the region of application may be only a few days to years. The entire trench construction project is intended to be completed with 24 months, thereby leaving this specific construction equipment on-site for as much as 3 construction seasons, providing breaks for inclement weather. Overall, the equipment used to construct Stresswalls and the time frame during which the equipment would be used are expected to result in only minimal noise impacts.

[23] John Babcock, Transwall Earth Retaining System, Ogden, UT

5.6.5.2 Traffic Impacts

Bottom-up construction raises traffic congestion concerns. Like cantilevered or mechanically stabilized earth retaining systems, these walls must be constructed at the base of a large excavation. Since braced excavations are difficult with the current configuration and operation of the UPRR, the excavation for a wall height of 22 feet would require 18 from the front face of the finished wall to the daylight point behind the wall. This large excavation has negative impacts including possible closure of adjacent parking lots and city streets. Based on these impacts, the Stresswall system is much less attractive than soil nailing or soldier pile and lagging walls.

5.6.6 Right-Of-Way Impact

Stresswall construction alone has no adverse impacts to adjacent right-of-way. The *the Draft Reno Railroad Corridor Environmental Impact Statement* indicates that the required underground easements for installation of grouted ground anchors is ample throughout the construction zone. Based on right-of-way impacts, these walls are applicable in Zone 1.

5.6.7 Aesthetics

Stresswalls may have facing materials installed. Paint, epoxy, color or texture impregnation and facades may be used for aesthetic treatments. Since these elements are precast and typically constructed of the same material, the uniformity in appearance of the completed construction is more pleasing than that of soldier pile and lagging. However, this added uniformity is more costly than the materials used in the soldier pile and lagging construction option. The additional architectural features, with the exception of the vegetation, can be constructed in the controlled environment of the precast yard, thus minimizing the costs and labor associated with modifications. Vegetation of these walls is accomplished through hydroseeding, a process that allows large areas to receive seed, fertilization, and environmental protection through a spraying process. Hydroseeding a Stresswall would cost approximately $2 per square foot of exposed wall.

5.6.8 Conceptual Calculations

Stresswalls are proprietary retaining structures; therefore, preliminary engineering is done with simplifications. The design concept for these walls is a combination of soldier pile and lagging with a gravity system. Based on design examples provided by the creator of the Stresswall system, the vertical segments can be constructed in three, 8-foot-tall units spaced at 8 feet on center with concrete louvered horizontal elements approximately 12 inches x 18 inches x 8feet-0 inches. These walls would require a base dimension of approximately 15 feet for a design height of 22 feet.

5.6.9 Cost[24]

Based on a maximum height of 22 feet and an average height of 15 feet, it is estimated that the Stresswall system could be constructed for $45/ft^2. Additional costs are associated with the construction of these walls in the region of the Rusty Spike substation, where large surcharges are imposed. A cost premium of approximately 30% would be required in that area.

5.6.10 History of Successful Application

The following section contains brief descriptions of similar successful projects using Stresswall systems.

Project:

Rail Road Grade Separation, El Cajon, California

Description:

The 1,500-foot-long wall along West Main Street is part of the East County trolley line extension from El Cajon to Santee Town Center. The retaining wall provides grade separation between the trolley line and surrounding improvements. The wall was designed to handle Cooper E-60 train surcharge loads. The layered concrete retaining structure is approximately 30 feet high with multi-tiered planters.

Owner:

San Diego, California Transit Authority

Contractor:

National Projects, Inc.

Specialty Contractor:

Spancrete of California

Project:

Theodore Roosevelt Dam Power Plant Road Retaining Wall, Roosevelt, Arizona

Description:

The project included the construction of a 40-foot-high near vertical retaining wall designed to support the power plant roadway. Rock anchors integrated with base tier counterforts were used to achieve slope stability.

Owner

U.S. Bureau of Reclamation

Contractor

Gibbons & Reed Company

[24] John Babcock, Transwall Earth Retaining Systems, Ogden, UT

Specialty Contractor

Tanner Prestressed Division

5.6.11 Advantages and Disadvantages of Stresswalls

Advantages:

- Complete precast concrete construction with no mechanical fasteners or metal in the backfill minimizes corrosion concerns.
- No site forming or field concrete is required other than for the pier foundation, which eliminates cold joints and quality control problems within the structural wall.
- Efficient erection and backfill facilitated by large precast components.

Disadvantages:

- Predrilling large diameter holes through hard soils and boulder obstructions can be a significant cost component.
- A Stresswall system is highly pervious and not suitable for granular soils below the groundwater table.
- Excavation for placement of precast panels between louver columns can cause ground loss outside of excavations.
- A Stresswall system if not properly braced or tied back may undergo relatively large lateral movements that could be detrimental to adjacent structures and utilities.

5.6.12 Application

5.6.12.1 Area of Use

The areas of use most appropriate for the Stresswall system are the regions within Zone 1. However, with secant/tangent piles or slurry-diaphragm construction in the region of the groundwater table in Zone 2, Stresswalls may be designed to rest on top. Therefore, although most appropriate in Zone 1, in combination with other systems these walls may be used in Zone 1 or Zone 2.

5.6.12.2 Installation Procedures

The installation procedure mimics that of soldier pile and lagging walls; however these walls must be constructed from the bottom up. Therefore, the installation of these walls, although viable, is not a favored choice for applications in Zone 1 of the Reno Railroad Corridor.

5.6.12.3 Functional Performance

Functionally, these walls are anticipated to perform adequately as both braced excavation systems and permanent structural walls. These walls have been used successfully in many of the same applications as soldier pile and lagging walls. Additionally, these walls are not intended to be groundwater barriers and only the lowest region of the wall is adaptable to stop minor groundwater infiltration. Therefore, stand-alone Stresswalls are functionally restricted to Zone 1.

5.6.12.4 Materials and Equipment Availability

The components used for these wall systems are precast concrete elements. The design of these elements can include provisions to cast on-site or at a local precast yard. Fabrication of the precast elements and transportation to the project site in Reno are available, as is the equipment used in the installation. The availability of both equipment and materials makes this wall system a viable choice for the Reno Railroad Corridor Project.

5.6.12.5 Conclusions

Stresswalls would perform adequately, would resist bridge loads and are constructible in Zone 1. However, compared to cantilever walls, mechanically stabilized earth and soil nailing, the slower production rate of this technique precludes it from being a leading candidate for wall construction in the Reno Railroad Corridor.

6 Secondary Systems

Secondary systems, as described in many of the competing wall options, are used to optimize structural sections of the primary wall systems. Most walls systems are compatible with these additions. The following section discusses the two leading choices secondary lateral support systems, their advantages and disadvantages, and their suitability for use in the Reno Railroad Corridor.

6.1 Struts (Zone 2)

6.1.1 Methodology

The lateral load carrying capacity of the proposed wall systems may be increased by installing horizontal struts. Struts are connected to beams or wales (stiff members arranged horizontally and running parallel to the wall alignment) at the wall top, bracing against the opposite wall. They may be constructed of many structural materials including concrete or steel.

The adjacent walls transfer the lateral forces from the walls to the struts. The struts then transfer the load to the opposite wall through axial forces. Struts, as compression members, must be design to resist the axial loads and braced against lateral torsional buckling. Struts are typically installed incrementally with progression of the wall construction.

Figure 6-1 Alameda Corridor Strutted Trench

6.1.2 Applicability to Soil Conditions

Since struts are used in conjunction with wall systems, applicability to soil conditions was not discussed.

6.1.3 Abutment Related Issues

Since struts span the opening of the trench, and vertical clearance from the train rails to the bottom of the bridges is typically designed for minimum values, struts are may not be applicable at the bridge locations. Some proposed structure types would not allow for installation of these strut directly under the superstructure. However, consideration may be made for use of struts contiguous to bridge superstructures or between structural elements of pre-cast bridges.

6.1.4 Duration of Construction

Since struts are pre-manufactured and installed during the excavation process, the impact to construction duration is minimal compared to the trench wall installation. However, mass excavation may be hindered by low vertical clearances when struts are first installed. It is anticipated that the struts would be installed at an approximate rate of five per day. Since these struts would be spaced at approximately 25 feet on center, the actual production rate would be limited by the production rate of the wall.

6.1.5 Traffic and Noise Impact

Noise impacts from the installation of struts is nominal. These struts are pre-manufactured and placed between the walls with a crane. Once placed, the struts are installed. Prefabrication and prompt installation virtually eliminate noise impacts. The only other impact is due to transportation.

Struts would be pre-manufactured and transported to the job site in 54-foot-long segments. These segments, loaded on trucks, may require detours and short-term, temporary lane closures that could be scheduled during off-peak traffic.

6.1.6 Right-Of-Way Impact

The greatest advantage of struts over the other choice (grouted ground anchors) relates to right-of-way impacts. Struts pass between the vertical trench walls and result in no right-of-way impacts to adjacent property.

6.1.7 Aesthetics

Another major difference between the two competing lateral support systems is their final installed appearance. Struts will be approximately 3.5 feet in diameter and can be constructed in rectangular, circular, elliptical, or other structural sections. The fact that struts are exposed leaves them vulnerable to aesthetic criticism. Additionally, concerns regarding the psychological impacts from bands of shadows on train rails must be examined to fully understand their effect on the train operator. Aside from psychological impacts, struts can be constructed with architectural treatments, including shape, color, texture, and lighting. These additions may add more than 10 percent to the total fabrication cost of each strut.

6.1.8 Conceptual Calculations

Determination of the loading criteria for the strut design may be extrapolated from similar projects. In previous applications of struts in the Alameda Corridor project of Southern California, the design criteria included specifications for loading and positioning. In the Alameda Corridor project, Union Pacific stipulated that the struts must carry an additional vertical load of 1,000 lbs/ft and catastrophic failure must be avoided if one strut is removed. It is assumed that the removal specification is to provide for accidental dislodging of a single strut during a derailment. This criterion was also applied to the strut design for the proposed Reno Railroad Corridor.

Based on spacing of approximately 25 feet on center, lateral loading applicable to Zone 2, and the design criteria specified in the Alameda Corridor, a circular hollow prestressed sections approximately 3.5 feet in diameter would be sufficient.

6.1.9 Cost

Based on the prestressing force and strut configuration, each strut would cost approximately $7,500[25]. Applying this cost to the trench section in Zone 2 the total cost would be $1.2 million.

6.1.10 History of Successful Application

A similar project in which struts have been used successfully is the Alameda Corridor Transportation Authority's, Alameda Corridor project in Los Angeles, California. At the Alameda Corridor, struts were employed to resist lateral forces from hydrostatic pressure, active earth pressure, and bridge structures. These struts were precast, prestressed octagonal concrete compression members. They were installed with tolerances within fractions of an inch.

6.1.11 Advantages and Disadvantages of Struts

Advantages:

- No additional right-of-way required behind the primary wall system.
- Quickly installed.
- Prefabricated members are produced with better quality control.

Disadvantages:

- Impairs mass trench excavation during construction operations.
- Aesthetic and psychological impacts from bands of shadows on rails, possibly affecting train operator.
- Limits access to trainway in case of emergencies.
- Cannot be constructed at shallow ends of the trench due to vertical clearance restrictions.

6.1.12 Application

6.1.12.1 Area of Use

Struts are applicable throughout the middle portion of the depressed trainway. Interfering with vertical clearance, struts are prohibited at the ends of the trench.

[25] Pomeroy Corporation, Petaluma California.

6.1.12.2 Installation Procedures

The installation procedure for struts is simple and swift. Struts are transported to a staging area with a crane. The crane positions the strut for installation into precast notches along the top of the wall. The strut is placed and the crane cables are removed. These procedures are repeated for each strut. The struts will be placed at approximately 25 feet on center, allowing the trench to be excavated to 10 feet below original grade. These procedures are advantageous for construction of the Reno Railroad Corridor.

6.1.12.3 Functional Performance

As seen with similar projects, external struts function adequately for the loads imposed by hydrostatic and active earth pressures, additional surcharge loads, and temporary construction procedures. Concerning the functional performance of struts, this technique would be adequate throughout the proposed project.

6.1.12.4 Materials and Equipment Availability

The materials and equipment used in the manufacture and installation of struts are readily available and do not preclude use of struts in the Reno Railroad Corridor.

6.1.12.5 Conclusions

Use of struts throughout the middle section of the trench, where minimum vertical clearance would be satisfied, would eliminate many property easement and acquisition concerns and provide a safe, cost effective, and efficiently installed solution for a secondary lateral support system applied to the primary wall techniques to be used in the depressed trainway.

6.2 Grouted Ground Anchors (Zone 1 or 2)

6.2.1 Methodology

The grouted ground anchor is a prestressed structural element designed to transmit tensile wall loads to the surrounding soil or rock. The elements are installed into a hole, which is then filled and pressurized with grout. The basic components of a grouted ground anchor include 1) the bond length, 2) the free stressing (unbonded) tendon length, and 3) the anchorage or wall connection. The bond length is the grouted length of a steel bar or strand tendon that bonds to the soil/rock and transfers the load into the ground. The bond length should be located far enough behind the critical failure surface of the retained soil/rock mass so that the supported mass is stable. The failure surface is frequently evaluated as a Rankine wedge with some modifications. The free stressing tendon is the unbonded tendon length free to elongate elastically and transfer the resisting force from the bond length to the wall. A bondbreaker is a smooth plastic sleeve placed over the free stressing tendon to prevent the tendon from bonding to the surrounding grout. The wall connection normally consists of an anchor head, bearing plate and trumpet. The wall connection transfers the tensile force from wall to the ground surface or prestressing force to the support structure.

Drilling methods for the grouted ground anchors include rotary, percussion, rotary/percussion, and auger drilling. Grouted ground anchor lengths, inclination, and location depend on the type of anchor, wall height, estimated earth pressures, utility locations, depth to groundwater, and number of rows of anchors.

Once the anchors are installed, proof testing is performed on each anchor by staged application to typically 133% of the design load. After testing is completed, typically the anchor is locked off at 75% to 110% of the design load.

Figure 6-2 Tieback Anchorages

6.2.2 Applicability to Soil Conditions

Grouted ground anchors can be constructed within the outwash materials and below the groundwater table present in Reno. Dewatering is not required for anchor installation below the groundwater table. However, seepage through the installation holes has been problematic in similar projects. As with all permanent ground anchors, and consistent with the geotechnical investigation, use of corrosion-resistant systems is recommended. Boring results show one sample at a depth of 6.5 feet with corrosivity potential. The recommendations from the geotechnical team include the use of Class I corrosion protection.

In addition, the use of grouted ground anchors is typically adequate for wall systems that may tolerate long-term deflections. Wall systems employing tiebacks will generally deflect away from the retained soil over time. This deflection, if planned for may be tolerated in Reno.

6.2.3 Abutment Related Issues

Grouted ground anchors at the bridge locations may increase the lateral capacity of the abutment structures. Grouted ground anchors are commonly used in retrofit schemes to activate more soil behind the abutment to help absorb seismic energy. The presence of grouted ground anchors has positive implications at the bridge abutments. Use of grouted ground anchors allows for shallow foundations and effectively reduces abutment construction costs.

6.2.4 Duration of Construction

Tieback anchors, constructed in the glacial till of the City of Reno, may be installed at a rate of four to six per shift per rig. Spacing these supports at approximately 8 feet on center, the contractor could complete 48 linear feet of wall in one day (one row). Each of these operations can be completed during other operations of the project and should not negatively impact productivity.

6.2.5 Traffic and Noise Impact

Since tieback anchors require drilling, noise impacts to nearby businesses and residences are more significant than those imposed by strut construction. However, like all the other drilling operations proposed at the Reno Railroad Corridor job site, the noise produced during tieback installation is minimal compared to the typical noise generated on local streets by existing traffic. One of the largest advantages to grouted ground anchors is the minimal traffic impacts imposed on local streets.

The equipment for installing grouted ground anchors can be confined to only the trench construction right-of-way. In addition, the materials used can be stockpiled on the location. The scheduling of the transportation of the materials can be adjusted to occur during off-peak hours. Therefore, traffic impacts from grouted ground anchors would be minimal.

6.2.6 Right-Of-Way Impact

The largest concern with grouted ground anchors is the potential for right-of-way impacts. The installation of tieback anchors necessitates maintenance and construction easements beyond those used for struts. When an underground obstruction breeches a property boundary, easements are inevitably required. The Reno Railroad Corridor is directly between businesses and right-of-way is scarce. In the areas of the project where nearby property boundaries or underground improvements are not a concern, tieback anchors are an appropriate solution. However, in the City of Reno vicinity, impacts due to tieback anchors may be significant.

In addition, future growth along the trench wall may include structures with below ground surface features, such as basements. Subsurface structures would impact the grouted ground anchor systems. Such construction challenges would require more coordination, expense, and time to overcome, and could require future property owners to be responsible for maintaining a secondary support for the wall in the region of their property.

6.2.7 Aesthetics

Since grouted ground anchors are installed into the supported soil and only the ends of the anchors are visible at the wall face, aesthetics is not an issue. Tieback anchors would result in fewer aesthetic impacts than struts for the Reno Railroad Corridor.

6.2.8 Conceptual Calculations

Based on the geology at the Reno Railroad Corridor project site and the design specifications for grouted ground anchors, the maximum unbonded length required for the tallest wall in this application is 34 feet. The minimum bonded length is 15 feet. Therefore, the entire ground anchor length is approximately 50 feet. Each anchor is required to provide approximately 100 kips of tension. For typical wall construction, the recommended horizontal and vertical spacing of the anchors is approximately 8 feet.

6.2.9 Cost

Based on historical data and discussions with construction specialists, each grouted ground anchor would cost approximately $4,000. Based on this estimate and the spacing indicated above, the additional cost associated with the installation of grouted ground anchors is $62 per square foot of exposed wall surface.

6.2.10 History of Successful Application

The following section contains similar successful projects utilizing grouted ground anchors.

Project:

State Route 91, Brigham City, Utah

Description:

The project involved constructing a 600-foot-long wall with a maximum height of 40 feet in a steep canyon. The subsurface conditions included large quantities of cobbles and boulders. The wall design incorporated soil nails and shotcrete (to temporarily stabilize the slope) and corrosion protected grouted ground anchors, cast-in-place wales, and precast panels (to provide the long term support of the cut). The system eliminated the need to install soldier beams.

Owner

Utah Department of Transportation and FHWA

Contractor

Le Grand Johnson Construction Company

Specialty Contractor

Schnabel Foundation Company

6.2.11 Advantages and Disadvantages of Grouted Ground Anchors

Advantages:

- A grout-anchored wall can resist relatively large horizontal earth pressures.
- Design procedures are well established.
- An unobstructed working space can be achieved.
- Quality assurance is achieved through proof testing.
- The effects of temperature change are less pronounced with ground anchors than with exposed bracing.
- Construction of walls supported by grout anchors requires specialty contractors and equipment.

Disadvantages:

- Underground easements would be required.
- Future construction including subsurface features is difficult.
- Anchors may be difficult construct where underground utilities are present.

6.2.12 Application

6.2.12.1 Area of Use

Grouted ground anchors are constructible throughout the entire trench alignment and would perform adequately above the groundwater table, below the groundwater table, and at bridge abutments. However, installation would necessitate movement of underground utilities and easements. Grouted ground anchors are suitable for use in all Zones of the Reno Railroad Corridor.

6.2.12.2 Installation Procedures

The installation procedures require drilling, installation, and grouting of prestressing strands. After the grout cures, the strands are tensioned with special hydraulic jacks. Specialized contractors with conduct the installation of grouted ground anchors.

6.2.12.3 Functional Performance

Functionally, tieback anchors have been used successfully on similar and more complex projects throughout the world for many years. Given the room to construct, there is no doubt tieback anchors would perform adequately for the proposed project. However, installation of grouted ground anchors below the groundwater table introduces concerns of leakage. These anchors have been proven problematic on other projects below water. Grouted ground anchors are one of the largest risks of being able to maintain a watertight system.

6.2.12.4 Materials and Equipment Availability

The materials and equipment for grouted ground anchors are common, and do not increase the costs or limit the field of bidders. However, specialized equipment, and skilled labor are available and can provide a more effective solution. Material and equipment availability enhances the consideration for grouted ground anchors to be considered as an different secondary lateral support system.

6.2.12.5 Conclusions

Grouted ground anchors provide unobstructed access to the trench, enabling quick mass excavation and limiting vertical clearance problems within the UPRR right-of-way. Additionally, grouted ground anchors do not pose psychological impacts to train operators. However, possible leakage concerns reduce their attractiveness for use below groundwater. With these limited impacts combined with prompt installation, enable a recommendation of grouted ground anchors for consideration in the Reno Railroad Corridor.

7 Inapplicable Methods

In examining the varied construction methods, the following options were determined inapplicable to trench construction in the Reno Railroad Corridor. Accompanying each method is a brief description of the fatal flaw that eliminated that particular option from consideration:

7.1 Ground Freezing

The scope of the document is to discuss, in detail, the viable different permanent systems available to function as trench walls. Ground freezing is a construction method that aids in the control of groundwater during excavation. Ground freezing is not a permanent wall system, therefore is not in the scope of the Wall Analysis Report.

Additionally, the high permeability of the soil stratum in the region of the proposed trench eliminates ground freezing as a practical construction aid for the proposed wall options.

Therefore, the requirement of fulfilling the scope of the document and offering viable construction methods eliminates ground freezing from detailed discussion.

7.2 Sheetpiling

Although a permanent wall and positive groundwater barrier, sheetpiling cannot be practically or reliably driven or vibrated in cobbled soil. The preclusion of vibration and driving makes a sheetpile wall inapplicable in all cases other than slurry-diaphragm construction.

It is possible to embed sheetpiles into the slurry excavation of a slurry-diaphragm wall. The impacts, advantages, and disadvantages of its use with a slurry-diaphragm wall are similar to those of precast panels. In addition, the use of sheetpiling in slurry-diaphragm walls is not typical of sheetpile wall construction; rather, it is a variation of slurry-diaphragm wall construction. Therefore, for the purposes of this report, sheetpile walls are unsuitable.

7.3 Deep Mixing

In consideration of deep mixing, preliminary investigation and consultation with specialty contractors purported its feasibility. However, through further geotechnical study and consultation with deep mixing experts is was determined that the initial supporters of the soil mix method did not fully understand the severity of the boulder issue which renders deep mixing untenable this project location.

8 Results (Wall Systems)

This report discusses the advantages, disadvantages, and applicability of thirteen independent wall systems. With each discussion topic, these wall options were challenged against project criterion that determined their applicability to the Reno Railroad Corridor project. With the large number of proposed options for the wall systems in this project, it was necessary to develop screening criteria to narrow these options to the most practical, feasible and economic wall type. A summary of each proposed wall method, segregated by region of applicability, is presented in the chart below.

Wall Type	Project Criterion									
	Applicability to Soil Conditions	Groundwater Control	Abutment Related Issues	Duration of Construction	Traffic and Noise Impact	Right Of Way Impact	Aesthetics	Cost	History of Successful Application	Application
Global Methods (Zone 1 or Zone 2)										
Slurry-Diaphragm Walls	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Jet Grouting	✓	✓	X	✓	✓	✓	X	✓	✓	✓
Permeation Grouting	X	✓	X	X	✓	✓	X	X	X	X
Secant/Tangent Piles	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Above Groundwater Methods (Zone 1)										
Cantilever Walls	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Mechanically Stabilized Earth Walls	✓	X	✓	✓	✓	✓	✓	✓	✓	✓
Micropile Walls	✓	X	✓	X	✓	✓	✓	X	✓	✓
Soil Nail Walls	✓	X	X	✓	✓	✓	✓	✓	✓	✓
Soldier Piles and Lagging	✓	X	✓	X	✓	✓	✓	✓	✓	✓
Stresswall System	✓	X	✓	X	✓	✓	✓	✓	✓	✓
Inapplicable Methods										
Ground Freezing	X	X	✓	✓	✓	✓	✓	✓	✓	✓
Sheetpiling	X	✓	✓	✓	X	✓	✓	✓	✓	X
Deep Mixing	X	✓	✓	✓	✓	✓	✓	✓	✓	X

Figure 8-1 Screening Criteria

In the chart above, each wall system is listed against the selection criteria and marked with a check (✓) for criteria that are satisfied and an "**X**" for criteria that are not satisfied.

Three wall types, examined for application above the groundwater table only, satified all selection criteria, 1) cantilever walls, 2) mechanically stabilized earth walls, and 3) soil nail walls. Below the groundwater table, two wall types satisfied all selection criteria, slurry-diaphragm walls and secant/tangent pile walls. The final recommendations test these wall types against costs and production rates.

The following production rates are based on the preliminary estimates obtained from specialty contractors and/or field experts in each of the construction methods. For regions above the groundwater table, a total of 180,000 ft^2 of wall is required. The regions below the groundwater table require 260,000 ft^2. Using double shifts at two headings, construction durations were developed.

Wall Type	Rate (ft^2/shift)	Remarks
Global Methods (Zone 1 or Zone 2)		
Slurry-Diaphragm Walls	800[1]	
Jet Grouting	135[2]	Based on 2 row of columns
Permeation Grouting	189[2]	Based on 2 row of columns
Secant/Tangent Piles	190[3]	
bove Groundwater Methods (Zone 1)		
Cantilever Walls	1,067	
Mechanically Stabilized Earth Walls	1,800[4]	
Micropile Walls	58[5]	
Soil Nail Walls	1,000[5]	May be increased 2 or 3 times depending on access and schedule
Soldier Piles and Lagging	215[3]	
Stresswall System	215[4]	

Figure 8-2 Production Rates

[1] Michael Pagano, TREVI ICOS Corporation, Morgan Hill, CA
[2] Mark Doehring, Kleinfelder, Reno, NV
[3] William Fishetti, PE, Malcolm Drilling Company, Vista, CA
[4] John Babcock, Transwall Earth Retaining System, Ogden, UT
[5] Donald A. Bruce, PhD, C. Eng, FICE, GEOSYSTEMS LP, Venetia, PA

Through collaboration with the specialty contractors, field experts, and published construction cost data from Caltrans, estimated costs were established. These wall costs, neglect the secondary lateral support systems (struts or grouted ground anchors) as required for each wall type. For final estimate purposes, the appropriate secondary support system must be selected and additional costs must be added.

Wall Type	Wall Cost	Remarks
Global Methods (Zone 1 or Zone 2)		
Slurry-Diaphragm Walls[1]	$70/ft²	
Jet Grouting[2]	$80/ft²	based on 2 rows of columns
Permeation Grouting[2]	$71/ft²	based on 2 rows of columns
Secant/Tangent Piles[3]	$107/ft²	
Above Groundwater Methods (Zone 1)		
Cantilever Walls[4]	$35/ft²	
Mechanically Stabilized Earth Walls[4]	$42/ft²	
Micropile Walls[5]	$67/ft²	
Soil Nail Walls[5]	$25/ft²	
Soldier Piles and Lagging[6]	$87/ft²	Included required anchors
Stresswall System[7]	$45/ft²	

Figure 8-3 Construction Costs

[1] Michael A. Pagano, P.E., TREVI ICOS Corporation, Morgan Hill, CA
[2] Donald A. Bruce, PhD, C. Eng, FICE, GEOSYSTEM, LP, Venetia, CA
[3] William Fishetti, PE, Malcolm Drilling Company, Vista, CA
[4] Caltrans Historic Data 1996 through 1999
[5] Ron Chapman, Schnabel Foundation Company, Walnut Creek, CA
[6] Caltrans Historic Data 1996 through 1999
[7] John Babcock, Transwall Earth Retaining System, Ogden, UT

9 Results (Invert)

This report discusses the advantages, disadvantages, and applicability of three independent invert systems. With each discussion topic, these invert options were challenged against project criterion that determined their applicability to the Reno Railroad Corridor project. With the diverse options for the proposed invert system in this project, it was necessary to develop screening criteria to narrow these options to the most practical, feasible and economic invert type. A summary of each proposed invert method is presented in the chart below.

Invert Type	Project Criterion: Applicability to Soil Conditions	Groundwater Control	Duration of Construction	Traffic and Noise Impact	Cost	History of Successful Application	Application
Global Methods (Zone 1 or Zone 2)							
Jet Grouting	✓	✓	✓	✓	✓	✓	✓
Permeation Grouting	X	✓	X	✓	✓	X	X
Cast-In-Place Concrete Slab	✓	✓	✓	✓	✓	✓	✓

Figure 9-1 Screening Criteria

In the chart above, each invert system is listed against the selection criteria and marked with a check (✓) for criteria that are satisfied and an “X” for criteria that are not satisfied.

Only two invert type satified all selection criteria, jet grouting and cast-in-place concrete slab. However, as noted in the report, these methods have a symbiotic relationship and function better as a joint application. Since the combination of these methods is the only viable option examined in this report, further examination of production rates and costs is not necessary for recommendation.

10 Conclusions and Recommendations

WALL SYSTEMS

Conclusions: The findings, summarized above, are refined by eliminating costly or slower constructed selections. Each region was examined independently. Those eliminated from contention in regions above the groundwater table due to production rates are 1) soldier piles and lagging, 2) Stresswalls and 3) micropiles. The remaining above groundwater applications (cantilever walls, mechanically stabilized earth, and soil nailing[not applicable at bridge abutments]) are all within acceptable limits with regard to cost and production rates.

Below the groundwater table, only two choices fulfilled the selection criteria, slurry-diaphragm walls and secant/tangent piles. Based on the best production rates and most economical solution, slurry-diaphragm walls are the preferred alternative.

Recommendations: Based on production rates, costs, and functional ability, the wall systems recommended to be most suitable for use above the groundwater table are: 1) cantilever walls, 2) mechanically stabilized earth and 3) soil nailing[not applicable at bridge abutments].

The wall system recommended to be most suitable for use below the groundwater table is slurry-diaphragm walls.

INVERT SYSTEMS

Conclusions: The findings, summarized above, illustrate a clear recommendation for an trench invert system. The preferred system for the Reno Railroad Corridor is a combination of two techniques, name jet grouting to construct a temporary groundwater barrier in preparation for a permanent invert system, a cast-in-place concrete slab. In addition, it was found that permeation grouting, while an adequate technique for creating a groundwater barrier serves better as a remediation method for localized seapage.

Recommendations: Based on production rates, costs, and functional ability, the invert system recommended to be most suitable for is jet grouting to temporarily seal the trench for the installation of a permanent solution, namely cast-in-place slab.

APPENDIX A

11 Peer Review Team

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APPENDIX B

14 Specialty Contractors

(Note: Specialty contractor information was provided solely by the following companies, and the claims made are theirs)

Condon Johnson & Associates, Inc.

Condon Johnson was founded in 1974 as a foundation engineer. Condon Johnson is known for innovative solutions for foundation engineering and excavation support. Condon Johnson's expertise includes deep mixing (using the GeoJet system), excavation shoring, drilling, grouted ground anchors, soil/cement columns (DMM) and cast in-place drill hole columns (CIDH).

Condon Johnson & Associates, Inc. Corporate Office
1840 Embarcadero
P.O. Box 12368
Oakland CA 94604
Telephone 510-534-3400
Fax 510-534-3421

Condon Johnson & Associates, Inc. – Los Angles Office
11040 Santa Monica Blvd., Suite 300
Los Angles, CA 90025
Telephone 310-575-1600
Fax 310-575-1606

GEO-CON

Geo-Con's corporate headquarters is in Monroeville, Pennsylvania. Regional offices are located in New Jersey, Florida, Texas, Pennsylvania and California. Geo-Con specialized technologies and services include construction of vertical barriers for groundwater control; insitu soil mixing and treatment; ground improvement; Deep Soil Mixing (DSM) soil mix walls and Vertical Earth Reinforcement Technology (VERT) – temporary and permanent retaining wall systems.

Geo-Con Corporate Headquarters
4075 Monroeville Boulevard
Corporate One, Building II, Suite 400
Monroeville, PA 15146
Telephone 412-856-7700

Geo-Con - California Office
3039 Kilgore Road
Rancho Cordova, CA 95670
Telephone 916-858-0480

Hayward Baker

Hayward Baker Inc. is one of the leading specialty contractors in Ground Modification^{sm} in North America. In recent years, a number of special techniques have been developed to "manufacture" predictable and dependable ground material, either in preparation for new construction or to correct problems that threaten existing structures. The techniques they use range from various methods of grouting, including chemical, cement, compaction, jet and Soilfrac^{sm}, to Vibro-Compaction (sometimes referred to as Vibroflotation), Vibro-Replacement and Vibro Displacement Stone Columns, Vibro Concrete Columns, Dynamic Deep Compaction^{tm}, soil and rock anchors, soil nails, minipiles, soil mixing, slurry trench cut-off walls and injection systems for expansive clays.

Hayward Baker Corporate Headquarters
1130 Annapolis Road, Suite 202
Odenton, Maryland 21113
Telephone 410-551-8200
Fax 410-551-1900

Hayward Baker – California Office
1780 Lemonwood Drive
Santa Paula, CA 93060
Telephone 805-933-1331
Fax 805-933-1338

Layne Christensen Company

Layne Christensen Company is a multi-faceted company that has been involved in subsurface development since the 1880's. The company is involved in a full spectrum of solutions for foundation engineering and civil construction. Their expertise involves micro-piles, auger cast piles, tie backs, soil nails, chemical grouting, jet grouting, underpinning, and ground freezing. They also have experience in geotechnical monitoring instrumentation including piezometers, extensometers, and inclinometers.

Layne Christensen Company Corporate Headquarters
1900 Shawnee Mission Parkway
Mission Woods, Kansas 66205
Telephone 800-407-4449
Fax 913-362-0133

Layne Christensen Company
W229 N5005 DuPlainville Road
Pewaukee, Wisconsin 53072
Telephone 414-246-4646
Fax 414-246-4784

Malcolm Drilling Company, Inc.

Malcolm Drilling was founded in 1962 and is currently the largest foundation contractor in North America. The company specializes in the elements of excavation support system construction from drilled shafts, Stresswalls, lagging, grouted ground anchors, underpinning, shotcrete, and tiedowns. Malcolm Drilling has a wide range of equipment including low overhead track rigs for tight access, and crane mounted drill rigs for large diameter, deep excavations. Malcolm's list of equipment includes the state-of-the-art Leffer Hydraulic Casing

Rotator used to construct bored piles with the full casing method under hard soil conditions.

Malcolm Drilling Company, Inc. Corporate Headquarters
200 Oyster Point Boulevard
South San Francisco, CA 94080
Telephone 650-952-9052
Fax 650-952-5542

Malcolm Drilling Company, Inc. - Northwest Division
7808 South 207th Court
Kent, WA 98032
Telephone 253-395-3300
Fax 253-395-3312

Malcolm Drilling Company, Inc. - Los Angeles Office
4926 North Azusa Canyon Road
Irwindale, CA 91706
Telephone 626-337-8131
Fax 626-337-5786

Malcolm Drilling Company, Inc. - San Diego Office
2125 La Miranda Drive
Vista, CA 92083
Telephone 760-727-0056
Fax 760-727-0881

Nicholson Construction Company

Nicholson is a recognized leader in geotechnical construction, specializing in ground improvement, ground treatment and ground support. As a Rodio Group company and subsidiary of London-based Stirling, Ltd., Nicholson had the experience and expertise to provide ground-related engineering and construction solutions throughout North America. From the construction of inner-city highway tunnels to deep excavations for building construction, Nicholson offers a full range of services including deep soil mix, jet grout, micro-piles, slurry-diaphragm walls and ground anchors required to provide a total solution.

Nicholson Construction Company Corporate Headquarters
P.O. Box 98
Bridgeville, PA 15017
Telephone 412-221-4500
Fax 412-221-3127

Nicholson Construction Company – California Office
P.O. Box 8790
Emeryville, CA 94662
Telephone 510-763-0365
Fax 510-763-0869

Schnabel Foundation Company

Transwall Patented Earth Retaining System

Transwall designs and provides precast Stresswall concrete elements for retaining wall construction throughout North America. Each Stresswall is engineered for specific site conditions and its intended use. By using the precast components numerous wall geometries and architectural styles are possible. Walls may be terraced for landscaping or vertical as required.

Transwall Patented Earth Retaining System – Utah Office
P.O. Box 3733
Ogden, UT 84409
Telephone 801-745-0902
Fax 801-745-4321

TREVI ICOS

The Trevi Group is a leading company in the field of special foundations, retention systems, water cut-off barriers, and soil treatment. The Trevi Group has over thirty years of experience at the international level, including North America. Normal daily activities carried out by the company in bored piles, slurry-diaphragm walls, micro-piles, ground anchors and jet grouting. The company has succeeded in facing challenging situations by being capable to diversify its own technologies to provide a total solution

TREVI ICOS Corporation US Corporate Office
250 Summer Street, 4th floor
Boston, MA, U.S.A.
Telephone 617-345-9955
Fax 617-345-0041

TREVI ICOS Corporation – California Office
P.O. Box 220
Morgan Hill, CA 95038
Telephone 408-782-8690
Fax 408-782-8691

Appendix C

Plan and Profile

Appendix D

Stability of Wall Construction

Appendix E

Seepage Memos